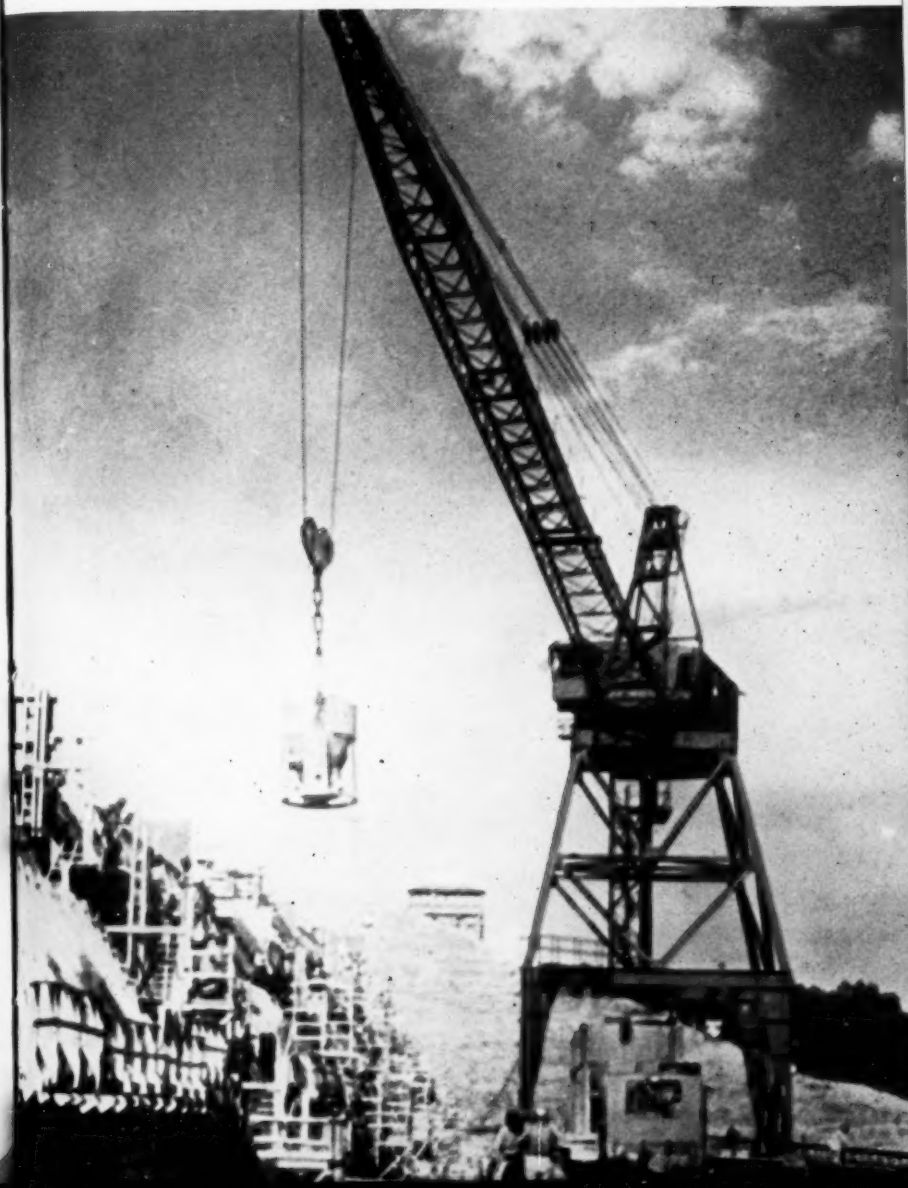
ACI News Letter

Vol. 22 No. 7 JOURNAL of the AMERICAN CONCRETE INSTITUTE March 1951



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News Letter Contents

March 1951

On the Cover—Concreting on the Clark Hill Dam by the Corps of Engineers. A 4-yd controllable bucket is being hoisted from the rail cars on the op of the trestle-way. See "Lean Mass Concrete Used for Interior of Dams," p. 553, for more details on gravity dam construction.

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First West Coast Meeting Successful

Splendid cooperation of an energetic and enthusiastic local committee made the 47th annual meeting in San Francisco a success for the 571 Members and friends of the Institute who attended.

Crowded technical sessions were relieved by trips to Chinatown, a cruise of San Francisco harbor (with a close-up view of Alcatraz) and a dinner for 350 in the Colonial Ballroom of the St. Francis Hotel—all with the compliments of the cement and concrete industry of the West Coast.

At a general luncheon attended by 275, President Frank H. Jackson announced the election to honorary membership of Harrison F. Gonnerman and Frank E. Richart, presented the annual awards and introduced the newly-elected officers and directors: Harry F. Thomson, President; Henry L. Kennedy, Vice-President; Frank Kerekes, G. L. Lindsay, I. E. Morris, and Walter H. Price, Directors.

Concurrent sessions worked smoothly despite some last minute changes in meeting places. Notable was the increased use of films to illustrate proper techniques of tilt-up construction, floor finishing, precasting and tests of prestressed beams. Emphasis was added to F. Thomas Collin's discussion of tilt-up construction when the rostrum did the reverse and landed at the feet of those in the first row amid a crash of glassware and tangled microphones. With splendid aplomb Mr. Collins continued his talk assured of a wide-awake audience.

The quiz session on Thursday saw Raymond E. Davis as a foil for many of the remarks of Chairman Scholer and members of the panel. Precast concrete and height of lifts in dams came in for rather lengthy discussion with surprisingly near agreement on the latter among users of both high and low lifts. H. J. Gilkey and Harland Edwards considered the value of the compression test as an indication of the quality of the concrete in a structure.

The annual research session included highlights of several new projects designed to study the properites and durability of concrete under varying test conditions. The summaries forecast several interesting papers increasing the fund of knowledge of concrete.

A more complete account of the convention will be published in the April JOURNAL.

Positions and Projects — ACI Members

Looking ahead to 1952

With the successful completion of the 1951 convention in San Francisco, ACI's Technical Activities Committee is already looking toward the 1952 meeting in Cincinnati and planning the program.

All those who have in mind offering papers for presentation at the 48th annual meeting should write to Institute headquarters no later than July 1, furnishing a synopsis which should make clear the intended scope of the paper and indicate features that, in the opinion of the author, will justify its inclusion in the program. Contributors should be prepared to have first drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and acceptance by Sept. 15, 1951, and final manuscripts by Jan. 1, 1952.

From the replies received and suggestions from other sources, TAC will select the papers to make up each convention session. Write Institute headquarters about that contribution *you* believe merits a place on the program.

Thomas appointed chief

John I. Thomas, with the Corps of Engineers for many years, has been appointed chief of the Atomic Energy Commission's division of engineering and construction at the Hanford Works, Richland, Wash. His most recent position was project engineer for Pine Flat Dam in California.

Willis J. Porter

Willis J. Porter, formerly materials and concrete engineer, Perkins Gravel Co., Sacramento, Calif., is now associated with O. J. Porter and Co., of the same city, in a similar capacity.

Deitrich joins consulting firm

Robert G. Deitrich, formerly structural field engineer for the Portland Cement Assn. in Maryland and Delaware, has become an associate of the consulting firm of J. L. Faisant and Associates, Inc., Baltimore, Md. A 1931 graduate of Rensselaer Polytechnic Institute, Mr. Deitrich was employed by the Baltimore District, U. S. Engineers, prior to his affiliation with PCA. He will be located in Baltimore where the firm acts as consultants on buildings and structures.

Calaveras Cement to expand storage

Four new cement storage silos with a total capacity of 75,000 barrels are being constructed for the Calaveras Cement Co., San Andreas, Calif., by Rust Engineering Co., Birmingham and Pittsburgh, Pa.

The silos are being constructed of reinforced concrete by the sliding form method. Each silo will be 32 ft in diameter and 94 ft high.

Turner Construction names officers

H. C. Turner, Jr., president of Turner Construction Co., New York City, has announced the election of George E. Horr as executive vice-president, a position which has not been filled in recent years, and of D. C. Andrews, F. B. Warren and E. K. Abberley of the New York office as vice-presidents, and S. H. Usher as senior assistant treasurer. H. A. Schroedel, vice-president in charge of the Philadelphia office, has been elected to the Board of Directors of the company.

Mr. Horr was formerly senior vice-president; Mr. Andrews and Mr. Abberley were formerly project executives and Mr. Warren was formerly senior assistant treasurer and assistant to the president.

Hogan lectures on prestressing

John Hogan, regional structural engineer of the Portland Cement Assn., New York, gave two lectures recently at the New York University College of Engineering on design details of prestressed concrete structures.

McWilliams elected director

William A. McWilliams has been elected director of the Delaware River Crossing

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McNary Dam
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division of the Delaware State Highway Department. He has been associate director for the last year, and was formerly chief engineer of the highway department, having been named to that post in 1946. He has been with the department since 1920, except for a four-year period of private employment.

Sanders reelected president

A. L. R. Sanders has been reelected president of the Illinois Section, ASCE, Chicago, Ill. Mr. Sanders is chief engineer for Hazelet and Erdal, consulting bridge engineers. Howard F. Peckworth, managing director of the American Concrete Pipe Assn., Chicago, has been named first vice-president of the society.

Cotter resigns company presidency

Carl H. Cotter, Rear Admiral, CEC, USN (Ret), president of Merritt-Chapman and Scott Corp., New York, N. Y., has resigned from that position to devote his time to activities connected with the national preparedness program. Admiral Cotter will be retained by Merritt-Chapman and Scott as a consultant.

Polivka appointed lecturer

J. J. Polivka, consulting engineer, Berkeley, Calif., and former research associate in civil engineering at the Engineering Materials Laboratory of the University of California, has been appointed visiting lecturer in architecture for the winter quarter in the department of art and architecture at Stanford University.

Willson re-elected to ASA committee

C. A. Willson, research engineer, American Iron and Steel Institute, New York, N. Y., has been re-elected to the executive committee of the Building Codes Correlating Committee of the American Standards Assn.

Lowden moves

S. W. Lowden, until recently district engineer for the California Division of Highways, at Bishop, Calif., is now district engineer at San Bernardino.

Lightweight concrete studies to help housing industry

Savings in materials and manpower through conservation and elimination of waste and inefficiency will be a determining factor in the volume of housing that can be built during the present emergency, Richard U. Ratcliff, Director of Research, Housing and Home Finance Agency, told the Southwest Research Institute Conference in Washington last month.

In describing the research program of the housing agency to assist industry in achieving such savings, he pointed out the potentialities for conservation through the use of alternate materials.

An HHFA sponsored project at the Southwest Research Institute seeks to find economical ways of building slabs and floor systems using local masonry materials. A project at the University of Toledo is directed to improving the stability of lightweight aggregate concrete blocks.

The National Bureau of Standards is studying reinforced lightweight concrete. The resulting elimination of dead weight can save at least 10 percent of the reinforcing steel required in certain applications, Dr. Ratcliff explained.

"The number of dwellings which we shall finally build out of the allotted resources," he said, "will be strongly influenced by the vigor with which we seek ways of saving materials and manpower and the extent to which we are able to raise the efficiency of production."

Highway problems discussed

How to build streets and highways in the present emergency drew the attention of more than 520 engineers and officials who attended the Third California Conference on Street and Highway Problems at the University of California in Berkeley.

Several speakers concluded that to stop building highways would be a blow to defense preparations. Besides the relationship of current transportation problems to national defense, the conference discussed toll roads, maintenance and traffic engineering.

The conference is presented annually by the Institute of Transportation and Traffic Engineering and University Extension, University of California.

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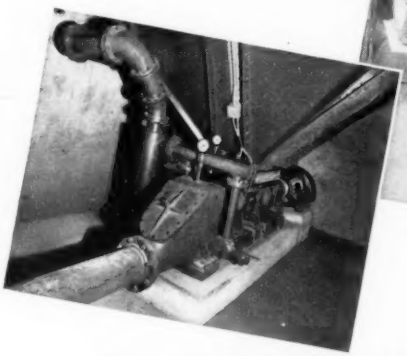

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Who's Who This Month

Arsham Amirikian

Well-known to JOURNAL readers, Arsham Amirikian, Head Designing Engineer, Bureau of Yards and Docks, Department of the Navy, Washington, D. C., submits a discussion of precast concrete that is particularly relevant to civil defense—"Precast Concrete Offers Protection Against Atomic Blast," p. 497. As chairman of ACI Committee 324, Precast Reinforced Concrete Structures, Mr. Amirikian has long been actively interested in precast construction.

After receiving his CE degree from Cornell University in 1923, he spent five years with private engineering and fabricating firms. In 1928 he joined the Bureau of Yards and Docks and has been with the Bureau for almost 23 years, beginning as an assistant structural engineer and being appointed to his present position in 1945.

For the past 12 years he has also headed the Bureau's special design section on buildings, floating structures, welding and protective construction. In the latter capacity, he has been associated with most of the important structures built by the Bureau, including the Navy's huge floating drydocks and gate caissons built of reinforced concrete.

In recent years he has devoted most of his time to the technique of precast concrete construction. He developed the thin-shell "hollow-rib" and "cellular" precast concrete framing systems utilized in a number of Navy shore and floating structures and is presently engaged in adapting these framing systems to protective construction.

R. F. Blanks and J. L. Gilliland

"False Set in Portland Cement," p. 517, is authored by R. F. Blanks and J. L. Gilliland, Chief, Research and Geology Division, and Head, Chemical and Cement Laboratory, respectively, Bureau of Reclamation, Denver, Colo.

Mr. Blanks, a past president of ACI, has contributed frequently to JOURNAL pages as well as being active on technical and administrative committees. Joining the Bureau in 1925, he has been with the Branch of Design and Construction since 1930. His outstanding work has been the development of laboratory and research techniques to solve diversified engineering problems. The engineering and geological laboratory under his direction has broadened its field to become one of the largest laboratories of its kind in the world.

Mr. Gilliland, also a contributor in past JOURNAL issues, has been on the Bureau staff since 1933. During this time he has worked on various aspects of cement research, especially the alkali-aggregate problem.

Henry J. Cowan

Henry J. Cowan, University of Sheffield, Sheffield, England, makes his first appearance in JOURNAL pages with "Direct Design of T-Beams," p. 533.

Since 1948 he has been a lecturer in civil engineering in charge of the concrete laboratory at Sheffield, as well as special lecturer in architecture. A graduate of Manchester University, Mr. Cowan received his MS there in 1940. He served with the Royal Engineers until he was wounded in Holland in 1945.

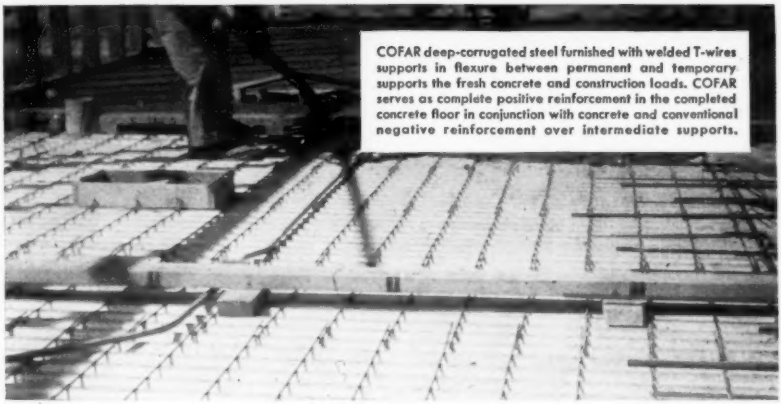
Returning to England, he was assistant engineer with Higgs and Hill, civil engineering contractors of London, and later senior designer in structural and mechanical development with a subsidiary of Associated Light Metal Industries. Prior to joining the University of Sheffield staff he was assistant lecturer at the University of Wales, Cardiff, Wales. He is the author of papers on concrete and reinforced concrete design in several English publications.

Federico Barona de la O

"Alkali-Aggregate Expansion Corrected with Portland Slag Cement," p. 545, is by Federico Barona de la O, Head, Technical Department of Materials, Hydraulic Resources Ministry, Mexico City, Mexico. An ACI Member since 1938, he is active on Committee 207, Properties of Mass Concrete.

In 1928 he was appointed concrete technologist at Rodriguez Dam and in 1933,

COFAR COmbination Form And Reinforcement



COFAR deep-corrugated steel furnished with welded T-wires supports in flexure between permanent and temporary supports the fresh concrete and construction loads. COFAR serves as complete positive reinforcement in the completed concrete floor in conjunction with concrete and conventional negative reinforcement over intermediate supports.

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Cold-drawn high-strength T-wires not over 6″ c.c. are welded to the deep-corrugated steel, and constitute temperature reinforcement in the slab, mechanical anchorage and positive shear transfer from concrete to steel.

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COFAR is concrete reinforcement, steel weight substantially equal to conventional reinforcement; *COFAR also is the concrete form.* Form sheeting as well as concrete runways are eliminated. COFAR comes detailed and cut to fit each job. Important material, labor, and time savings are made.

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head of the materials laboratories for Mexico's large reclamation program under the *Comision Nacional de Irrigacion.* In 1947 when the commission was raised to the rank of a federal ministry, he was appointed to his present position.

A graduate of the University of Mexico, he taught materials of construction at the Instituto Politecnico Nacional for 16 years and is the author of various papers on cement and concrete in the technical journals of Mexico.

Byram W. Steele

Methods of achieving both durability and economy in gravity dam construction are discussed in "Lean Mass Concrete Used for Interior of Dams," p. 553, by Byram W. Steele.

Mr. Steele is Special Assistant, Civil Works Division, Office of the Chief of Engineers, Department of the Army, Washington, D. C. An ACI Member since 1929 he is active on Committees 212, Admixtures; 620, Construction Joint Practice; and 621, Aggregate Selection, Preparation, Handling and Use.

Last November Mr. Steele completed 30 years of Federal service on dam design and construction—the first 16 years with the Bureau of Reclamation, the last 14 with the Corps of Engineers. He was in charge of storage dam design and materials investigations for the Bureau. His work with the Corps of Engineers has been along the same line.

As a special assistant, Mr. Steele acts as a consultant to district and division offices on problems in dam design and construction. In recent years he has spent considerable time on dam design investigations for foreign governments. He recently returned from Colombia and Venezuela where he was investigating damsites.

Economical, durable concrete for hydraulic structures has been a specialty with Mr. Steele since the construction of Hoover Dam when the first extensive mass concrete investigations were undertaken.

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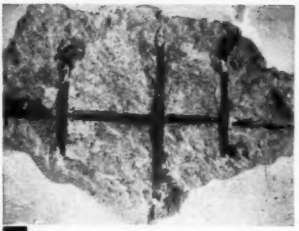
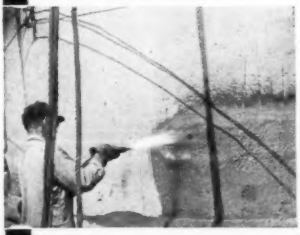


Typical example of spalling. Note corrosion of reinforcing rods exposed by disintegrating concrete.



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Honor Roll

February 1, 1950 to January 31, 1951

The Morgans have done it again! The combined efforts of father and son have produced a total of 70½ credits to lead the Honor Roll for the second successive year. Newlin D. Morgan of Illinois, 48½ credits, placed first; Newlin D. Morgan, Jr. of Wyoming, 22 credits, second; and Walter T. Daniels, District of Columbia, placed third with 11 credits.

All of ACI's more than 5200 Members should be grateful to these and the other men on the Honor Roll for their efforts in bringing in "new blood."

February 1 starts a new year for the Honor Roll and ACI could grow and make 1951 an outstanding year for introducing new members to our organization if all members could "spread the word" and take high places on the Honor Roll.

Newlin D. Morgan (Ill.)	**48½**
Newlin D. Morgan, Jr. (Wyo.)	**22**
Walter T. Daniels (D. C.)	**11**
Leon Venegas (La.)	**10**
Rafael Ruiz P. (Guatemala)	**9**
Henry E. Griset (N. C.)	**7**
R. H. Sherlock (Mich.)	**7**
E. I. Fiesenheiser (Ill.)	**6½**
Chester P. Siess (Ill.)	**6½**
George C. Alden (Calif.)	**5½**
A. Amirikian (Md.)	**5½**
Earl I. Brown (Ga.)	**5**
Raymond E. Davis (Calif.)	**5**
Charles G. Peller (Ind.)	**5**
Luis A. Pietri-Lavie (Venezuela)	**5**
Andrew Reti (Venezuela)	**4½**
Byram W. Steele (D. C.)	**4½**
E. W. Bauman (D. C.)	**4**
F. Thomas Collins (Calif.)	**4**
Phil M. Ferguson (Texas)	**4**
Samuel Hobbs (Calif.)	**4**
Frank H. Jackson (D. C.)	**4**
J. E. Jellick (Calif.)	**4**
Henry L. Kennedy (Mass.)	**4**
James A. McCarthy (Ind.)	**4**
Oscar A. Nunez (Venezuela)	**4**
Cesar Oliver-Rugeles (Venezuela)	**4**
Ivan M. Viest (Ill.)	**4**
James L. Atkinson (Calif.)	**3½**

Castor Segundo Goa (Venezuela)..... 3½
Wm. T. F. Hooper, Jr. (Ill.)..... 3½
Melvin W. Jackson (Ga.)..... 3½
S. Moore (Canada)..... 3½
Clayton M. Crosier (Kans.)..... 3
Ashby T. Gibbons, Jr. (Ga.)..... 3
Gerald K. Gillan (Mo.)..... 3
Emil A. Gramstorff (Mass.)..... 3
Walter N. Handy (Ill.)..... 3
M. D. Hicklin (Ore.)..... 3
G. Millsom (Canada)..... 3
George D. Youngclaus (Calif.)..... 3
Aloysius E. Cooke (Conn.)..... 2½
Rodolfo C. Garcia (Philippine Islands)... 2½
H. J. Gilkey (Ia.)..... 2½
Orlando W. Irwin (Ill.)..... 2½
W. H. Jacobs (Ill.)..... 2½
Jose Antonio Vila (Cuba)..... 2½
S. J. Warberg (Mich.)..... 2½
Ralph L. Barbehenn (N. J.)..... 2
Roberto Barillas F. (Guatemala)..... 2
Hugh Barnes (Calif.)..... 2
Sterling Lowe Bugg (Fla.)..... 2
W. Fisher Cassie (England)..... 2
J. J. Creskoff (D. C.)..... 2
Belmon U. Duval (Ohio)..... 2
Eddy N. Hernandez (La.)..... 2
J. Sterling Kinney (N. Y.)..... 2
George E. Large (Ohio)..... 2
William Lerch (Ill.)..... 2
F. J. Ochoa U. (Guatemala)..... 2
Quentin Rust (Calif.)..... 2
Thomas C. Shedd (Ill.)..... 2
Leroy A. Staples (La.)..... 2
J. Neils Thompson (Texas)..... 2
H. F. Thomson (Ill.)..... 2
A. G. Timms (D. C.)..... 2
J. F. Baxter (N. Y.)..... 1½
Oscar Benedetti (Venezuela)..... 1½
Anthony Henry Clark (England)..... 1½
Raymond E. Davis, Jr. (Calif.)..... 1½
Charles W. DeGroff (N. Y.)..... 1½
Harlan H. Edwards (Wash.)..... 1½
H. F. Gonnerman (Ill.)..... 1½
Ernst Gruenwald (N. Y.)..... 1½
George E. Hatch (Hawaii)..... 1½
Elmo C. Higginson (Colo.)..... 1½
Leo M. Legatski (Mich.)..... 1½
H. J. McGillivray (Fla.)..... 1½
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Roderick B. Young (Canada)..... 1½
J. B. Alexander (Calif.)..... 1
F. O. Anderegg (N. J.)..... 1
Edward L. Ashton (Ia.)..... 1
Eugene D. Ayer (Colo.)..... 1
F. Barona (Mexico)..... 1
Hans H. Bleich (N. Y.)..... 1
Woodrow L. Burgess (Ore.)..... 1
E. F. J. Clark (Canada)..... 1
Roy R. Clark (Ore.)..... 1
Kenneth W. Clayton (Australia)..... 1
W. S. Cottingham (Wis.)..... 1
M. A. Craven (New Zealand)..... 1
R. A. Crysler (Canada)..... 1
Harmer E. Davis (Calif.)..... 1
DeWitt Dieterich (Ill.)..... 1
Joseph Di Stasio (N. Y.)..... 1
A. H. Douglas (Canada)..... 1
Leonard E. Dunlap (Ill.)..... 1
H. B. Emerson (Ill.)..... 1
C. C. Everhart (Ia.)..... 1
Jesus R. Ferrer, Jr. (Philippine Islands)..1
Stanley Gordon Fisher (Canada)..... 1
Louis R. Forbrich (Pa.)..... 1
George W. Ford (Fla.)..... 1
Jose M. Franco-Cancio (Puerto Rico)..... 1
William E. Gabelman (Calif.)..... 1
Randall L. Gomien (Ohio)..... 1
Otto Graf (Germany)..... 1
Nathaniel Grant (Mo.)..... 1
Gilberto Guardia (Panama)..... 1
Horace P. Hamlin (N. Y.)..... 1
Robert J. Hansen (Mass.)..... 1
Terrell R. Harper (Texas)..... 1
W. L. Harrell (Texas)..... 1
Charles E. Hawke (Canada)..... 1
Harry H. Hawley (Ohio)..... 1
Philip W. Helsley (Calif.)..... 1
Bernard C. Herring (England)..... 1
Wallace E. Johnsen (Ind.)..... 1
William R. Johnson (Ore.)..... 1
Edgar R. Kendall (Nebr.)..... 1
Thomas B. Kennedy (Miss.)..... 1
Frank Kerekes (Ia.)..... 1
John C. King (Ohio)..... 1
K. V. Kini (India)..... 1
R. A. Kirkpatrick (N. Y.)..... 1
Blas Lamberti (Venezuela)..... 1
Demetrio Lausell (Puerto Rico)..... 1
David V. Lewin (Ohio)..... 1
Harry E. Lewis (N. Y.)..... 1
Alvin C. Loewer, Jr. (Pa.)..... 1
Winthrop E. Luke (Mass.)..... 1
Charles L. Luther (Ky.)..... 1
Ian Macallan (New Zealand)..... 1
Glenway Maxon (Wis.)..... 1
B. H. Maynard (Calif.)..... 1
Frank E. McClure (Calif.)..... 1
G. T. McCoy (Calif.)..... 1
J. H. Murdough (Texas)..... 1
J. A. Murlin (Texas)..... 1
James A. Murray (Mass.)..... 1
Amado B. Navoa (Philippine Islands)..... 1
George A. Norwood (Texas)..... 1
Carlos Ospina (Colombia)..... 1
Wm. Otcovsky (Ill.)..... 1
Kenneth K. Paget (Canada)..... 1
Jose Antonio Pardo B. (Mexico)..... 1
Dean Peabody, Jr. (Mass.)..... 1
Howard F. Peckworth (Ill.)..... 1
Stanley A. Phillip (Ill.)..... 1
Harry W. Piper (Md.)..... 1
J. D. Piper (Texas)..... 1
Dan H. Pletta (Va.)..... 1
Herman G. Protze (Mass.)..... 1
Theodore O. Reyhner (Colo.)..... 1
Paul F. Rice (Mich.)..... 1
Evan L. Richard (Australia)..... 1

Frank E. Richart (Ill.).....1
Horace B. Rickey, Jr. (La.).....1
Bernard L. Robinson (N. Y.).....1
Gilbert H. Rowe (S. C.).....1
Otto Sauermann (Ill.).....1
Emil Schmid (N. J.).....1
Wilfrid Schnarr (Canada).....1
C. H. Scholer (Kans.).....1
Elwyn E. Seelye (N. Y.).....1
E. R. Smallhorn (Canada).....1
Howard R. Staley (Mass.).....1
Kenneth G. Stevens (S. Rhodesia).....1
Herbert M. Stoll (Ill.).....1
M. Eugene Sundt (N. M.).....1
Pascal L. Tasio (N. Y.).....1
Wm. H. Thoman (Colo.).....1
E. W. Thorson (Colo.).....1
LeRoy A. Thorssen (Canada).....1
G. E. Troxell (Calif.).....1
Gregory P. Tschebatarioff (N. J.).....1
Saul Uhr (D. C.).....1
Maurice P. Van Buren (N. Y.).....1
F. E. Votaw (Mass.).....1
A. G. Watt (Pa.).....1
W. R. Waugh (D. C.).....1
C. A. Willson (N. Y.).....1
Merle D. Wilson (Ohio).....1
Arthur G. Wood (Colo.).....1
S. H. Woodard (N. Y.).....1
A. E. Wynn (S. Africa).....1
Aubrey D. Young (La.).....1
Edward F. Young (Minn.).....1
R. A. Zern (Pa.).....1

The following credits are, in each instance, "50-50" with another member.

Marcelo Abadia
James Adam, Jr.
Herbert C. Allen
Frederick G. Anderson
R. Howard Annin
Charles D. Babcock
Malcolm F. Baker
C. Merrill Barber
Joseph I. Bargar
George E. Barnes
Frank Baron
A. Allan Bates
K. P. Billner
G. L. Blanchard
A. E. Bond
Wallace G. Bond
Harry E. Born
J. G. Bragg
James E. Branch
Charles Britzius
Carlos D. Bullock
Ernest W. Burke
S. D. Burks
R. A. Burmeister
Harold J. Buttery
D. J. Cameron
Neal J. Campbell
Julian B. Carson
Harold Carter
S. J. Chamberlin
Jules P. Channing
J. Chubb
Ralph E. Coblentz
V. N. Connar
Miles K. Cooper
Roger H. Corbetta
William A. Crabb
A. E. Cummings
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Robert G. Deitrich
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Harry Delzell
R. G. Douglas, Jr.
Albert W. Dudley
John R. Dwyer
Harry Ellsberg
W. J. Emmons
H. C. Farmer
L. G. Farrant
Jorge Figuls
O. H. Fosswein
John D. Freeman
P. J. Freeman
Bengt Friberg
Bernardo Fuentes A.
Mario W. Gamalero
J. H. Gandhi
E. L. Gardner
A. H. Garmsey
Grayson Gill
A. T. Goldbeck
O. H. Gosswein
Alex H. Graves
Homer M. Hadley
Albert Haertlein
Theodore F. Hagerman
William H. Hall
W. C. Hansen
Harvey L. Harris
Roy Hawkins
John M. Hayes
Eivind Hognestad
W. A. Hohlweg
William N. Holway
Wilfred M. Honour
E. S. Horne
J. W. Hubler
Virgil L. Hughes
Maurice Hunt
Anthony Hunter
S. E. Hyde, Jr.
Raja A. Iliya
W. R. Ingram
C. E. Ireland
C. L. N. Iyengar
Arthur M. James
M. E. James
Bruce M. Johnson
J. G. Johnson
H. D. Jolley
Bernerd D. Keatts
J. W. Kelly
Clyde Kesler
W. D. Kimmel
W. P. Kinneman
Ralph W. Kluge
Theodore C. Knight
Jean H. Knox
M. C. Kolinski
Paul P. Kraai
Richard J. Kroc
W. G. Limbach
J. E. Lothers
Irvin H. Luke
I. D. MacKenzie
M. F. Maenaughton
James V. Mandia
Eastman M. Markell
William Marquez
W. T. McClenahan
Sam McCluer
C. T. J. McDowell
A. J. McElrath
Douglas McHenry
Harmon S. Meissner
F. N. Menefee
Willis T. Moran
E. J. Mueller
Glenn Murphy
V. S. Murray
A. E. Myers
William T. Neelands
Henry L. Neve
Walter Newland
Roy A. Nyquist
Raymond G. Osborne
John P. Ottesen
Douglas E. Parsons
Orley O. Phillips
Thomas Poli
Milos Polivka
James A. Polychrone
Chesley J. Posey
Leo F. Pratt
Walter H. Price
Charles J. Prokop
Raymond C. Reese
R. L. Reid
Ross M. Riegel
Guy Rinfret
Charles S. Rippon
Richard A. Roberts
A. Rodriguez-Delfino
Pedro Rodriguez
Ansel T. Rogers
Paul Rogers
A. W. Rohlwing
Mark R. Rosumny
Oswaldo Rovati B.
William H. Rowan
John A. Ruhling
Frederic Rusche
Nicholas R. Samaha
Robert L. Sanks
R. W. Sauer
D. M. Schmid
Axel Schulze
James M. Shivley
J. Morgan Smith
Myron M. Smith
Ernest L. Spencer
R. A. Spencer
R. W. Spencer
John J. Stedje
Frank C. Stewart
John D. Stites
Hale Sutherland
Ernest O. Sweetser
Thomas F. Taylor
L. W. Teller
Walter F. Tews
George P. Thigpen
J. Trueman Thompson
B. K. Thornley, Jr.
T. Thorvaldson
Paul M. Trueblood
Lee Turzillo
Lewis H. Tuthill
I. L. Tyler
Miguel Villa
E. H. Walker
Julius J. Warner
G. E. Warren
Willard W. Warzyn
Stewart F. Weikel
L. T. Willoughby
Clement T. Wiskocil
Francis P. Witmer
A. E. Wood
Bourdette R. Wood
H. D. Worthington

For the
COMPLETE CONVENTION STORY
see
APRIL NEWS LETTER

New Members

Washington, D. C., leads this month with 10 new member applicants; California is second with 8.

There were 72 new members approved for January, bringing ACI's total membership to 5243.

Alabama

Kilgore, A. L., (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., Demopolis, Ala.

Williams, H. A., (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., 2026 2nd Ave. N., Birmingham 3, Ala.

California

Bockemohle, C. L. A., (Indiv.) 256 S. Berendo St., Los Angeles 4, Calif.

Ekvall, John Carter, (St.) 1631 Oak View, Berkeley 7, Calif.

Hazard Concrete Products, Inc., (Corp.) P. O. Box 1951, San Diego, Calif.

Jacobs, Fred W., (Indiv.) 666 Mission St., San Francisco 3, Calif.

McNely, Warren E., (St.) Room 18 Engineering Bldg., Univ. of California, Berkeley 4, Calif.

Parker, Edward H., (Indiv.) 11943 Briarvale Lane, Studio City, Calif.

Robinson, William J., (Indiv.) P. O. Box 35, Goleta, Calif.

Smith, Yorke H., (Indiv.) 1818 Torrence St., San Diego 3, Calif.

District of Columbia

Bailey, Donald H., (Indiv.) 511 Harvard, N. W., Washington, D. C.

Baines, Bedford B., Jr., (St.) 716 Otis Place, N. W., Washington 10, D. C.

Brewer, Elijah, (St.) 1312 Girard St., N. W., Washington 9, D. C.

Chapital, Alden N., (St.) 752 Princeton Pl., N. W., Washington 1, D. C.

Cooper, Stanley A., (St.) 2229 13th St., N. W., Washington, D. C.

Dunnigan, Emmett, (St.) Howard University Engr. School, Washington, D. C.

Jennings, Clifton M., (St.) 3501 13th St., N. W., Apt. 43, Washington, D. C.

Johnson, Napoleon, (St.) Box 82, Howard University P. O., Washington, D. C.

Stamp, Jose Lester, (St.) Box 561, Cook Hall, Howard University, Washington 1, D. C.

Taliaferro, Will C., (St.) 211 Elm St., N. W., Washington, D. C.

Georgia

Gentsch, E. F., (Indiv.) Box 4197, Ga. Tech., Atlanta, Ga.

Illinois

Chang, Rodney Kam Jan, (St.) 1341-3 Third St., P. G. U., Champaign, Ill.

Weeks, Edward L., (Indiv.) 268 Walker Ave., Highland Park, Ill.

Indiana

Moore, C. J., (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., Greencastle, Ind

Kansas

Griffith, J. H., (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., Bonner Springs, Kans.

McCord, Hal H., (Indiv.) 409 Crawford Bldg., Topeka, Kans.

Kentucky

Mills, Billie C., (St.) 2335 W. Magazine St., Louisville 11, Ky.

Louisiana

Gunn, J. W., (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., 812 Gravier St., New Orleans 12, La.

Maryland

Cassell, James Robert, (Indiv.) 6112 Bertram Ave., Baltimore 14, Md.

Hench, Lynn H., (Indiv.) 5205 Andover Rd., Chevy Chase 15, Md.

Michigan

Arrighini, Artil, (Indiv.) 15160 W. Eight Mile Rd., Detroit 35, Mich.

New Jersey

Acken, Howard W., (Indiv.) Delaware and Raritan Canal, 342 Academy St., Trenton 8, N. J.

Schupak, Irving I., (Jr.) 9 McAdoo Ave., Jersey City, N. J.

New York

Elsenhans, G. E., (Indiv.) Universal Atlas Cement Co., 100 Park Ave., New York 17, N. Y.

Jernigan, I. C., (Indiv.) Asst. Dir. of Research, Lone Star Cement Corp., Hudson, N. Y.

Judge, Thomas Joseph, (St.) 20 West St., Green Island, N. Y.

Pritchard, L. R., (Indiv.) Chief Chemist, Lone Star Cement Corp., 100 Park Ave., New York 17, N. Y.

Robinson, Edward A., (Indiv.) The Ardsley, Garth Rd., Scarsdale, N. Y.

Ohio

Barnes, James R., (Indiv.) P. O. Box No. 206, Columbus 16, Ohio
Stephens, Eugene M., (Indiv.) The Flexicore Co., Inc., P. O. Box 825, Dayton 1, Ohio

Pennsylvania

Hetrick, L. E., (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., Nazareth, Pa.
Jenkinson, Robert B., (Indiv.) Corps of Engineers, Saltsburg, Pa.
McCoe, Jack C., (Indiv.) Silver Lane RFD, McKees Rocks, Pa.

Texas

Broad, John, (Indiv.) Box 494, Austin 63, Texas
Dodge, Lee, (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., P. O. Box 168, Sweetwater, Texas
Hailey, O. L., (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., 1407 Main St., Dallas, Texas
Tilley, W. H., (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., 815 Walker St., Houston 2, Texas

Virginia

Dillon, L. L., (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., P. O. Box 1107, Norfolk, Va.
Kendrick, W. J., Jr., (Indiv.) Div'n. Ch. Chemist, Lone Star Cement Corp., General Delivery, Cloverdale, Va.
Watson, Robert G., (Indiv.) 916 S. Buchanan St., Apt. 203, Arlington, Va.

Wisconsin

Loescher, Richard A., (St.) 365 Naymut St., Menasha, Wis.

Hawaii

Nakamura, Takashi (Jr.) 2464 Kanealii Ave., Honolulu 13, T. H.

Puerto Rico

Dionisi, M. A., (Indiv.) P. O. Box 8757, Fdez. Juncos Sta., Santurce, P. R.
Sarriera, Rafael E., (Jr.) Box 56, Catano, P.R.

Argentina

Belling, E., (Indiv.) Sup. Chemist, Cia. Argentina de Cemento Portland, Calle Reconquista 46 (R. 3), Buenos Aires, Argentina
Carey, W. O., (Indiv.) V. Pres., Cia. Argentina de Cemento Portland, Calle Reconquista 46 (R. 3), Buenos Aires, Argentina

Brazil

Laranja, F., (Indiv.) Ch. Chemist, Companhia Nacional de Cimento Portland, Avenida Presidente Wilson No. 164, Rio de Janeiro, Brazil

Canada

Sutherland, James B., (Indiv.) 2379 Columbia Ave., Trail, B. C., Canada

Cuba

Bramnick Werbin, Zacarias, (Indiv.) Calzada 355, Apt. 9, entre GyH Vedado, Havana, Cuba
Gelabert, Jose A., (St.) Neptuno 1223, esq. Ma on, Havana, Cuba
Rutzen Garbers, Gustavo, (St.) Ave. Santa Amalia No. 112, Arroyo Apolo, Havana, Cuba
Villa, Miguel, (Indiv.) Compania Cubana de Cemento Portland, Apartado 829, Havana, Cuba

Egypt

Bari, A. M. Abdel, (Jr.) 35 Abbasiah St., Cairo, Egypt

England

Bridges, George Percival, (Indiv.) "Dundonald," St. Michaels Ave., Bramhall, Cheshire, England

Japan

Itakura, Chuzo, (Indiv.) Hokkaido University, Faculty of Technology, North 12, West 8, Sapporo, Hokkaido, Japan

Panama

Weinrauch, Leopold Karl, (St.) Central Ave. No. 9129, P. O. Box No. 966, Colon, Rep. of Panama

Philippines

Koh, Filiciano V., (Jr.) 642 Benavides, Binondo, Manila, P. I.

Spain

Instituto Technico de la Construccion y del Cemento, (Corp.) Velazquez 47, Madrid, Spain (Jaime Nadal Aixala)
Nadal Aixala, Jaime, (Indiv.) Instituto Technico de la Construccion y del Cemento, Velazquez 47, Madrid, Spain

Uruguay

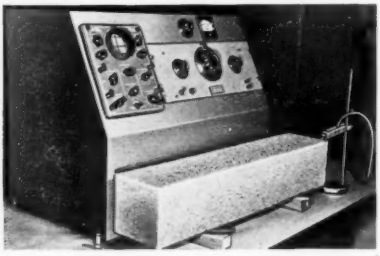
Buchholtz, H., (Indiv.) Chief Chemist, Compania Uruguaya de Cemento Portland, Calle Zabala 1338, Montevideo, Uruguay

Venezuela

Buzney, Anthony J., (Indiv.) Creole Petroleum Corp., c/o Gen. Engr. Group, Cabimas, Estado Zalia, Venezuela
Giordano, Alfonso, (Indiv.) Edificio M-D, Apartamento 1, Puente Hierro, Caracas, Venezuela

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.



Sound vibrations measure modulus of elasticity

Electro Products Laboratories have developed from the Portland Cement Association's basic design a new Electro Sonometer for determining the resonant frequency of any solid mass or material where strength is an important factor. It consists of two units, the control cabinet and the portable pick-up.

The control cabinet contains an amplifier supplying 18 watts of power at the driving unit which permits measurement of resonance frequency in solid masses weighing as much as 1500 lb. An oscillator measures resonant frequency from 20 cycles to 22 kilocycles. The phase relationship between the driving voltage at one end of the mass and various points within the mass can be determined and viewed on the screen of the cathode ray oscilloscope.

Experienced operators can determine flexural and torsional modes of vibration, and information from these tests can be used to determine Young's modulus of elasticity using the formula in PCA Bulletin No. 7. —Electro Products Laboratories, 4501 Ravenswood Ave., Chicago 40, Ill.

Testing machine

Redesigned to separate the loading and weighing units, a new 100,000-lb capacity testing machine has been announced by Baldwin-Lima-Hamilton Corp. The two-unit design prevents transmission of load shocks to the indicator and keeps the operator out of range of flying or falling particles from breaking specimens. The new machine is designed primarily for testing 2-in. cubes and 3 x 6-in. cylinders.—Testing Equipment Dept., Baldwin-Lima-Hamilton Corp., Philadelphia 42, Pa.

Plastic coatings control corrosion

The use of vinyl plastic coatings for the control of corrosion on exteriors of steel, concrete, brick and tank linings is described in an illustrated bulletin released by Casey and Case Coating Co. Coatings are of an inert thermoplastic resin base. The manufacturer states that they may be used in contact with food processing and are effective against caustic sterilizing or cleaning agents. Specific properties are listed, a chemical resistance chart is included, as well as application instructions.—Casey and Case Coating Co., Dept. AC, P.O. Box 151, Maywood, Calif.

Precast prestressed slabs

Precast prestressed concrete slabs for floors and roofs are described in a new 8-page bulletin released by Flexicore Co., Inc. The well-illustrated booklet explains how prestressing is utilized in the hollow core slabs. A simplified load chart for 10 to 25-ft spans is included. Detailed drawings show how Flexicore slabs can be used in various types of construction. Electrical, plumbing and heating details are shown. —Flexicore Co., Inc., 1932 E. Monument Ave., Dayton 1, Ohio.

Hydraulic jacks

Applications of hydraulic pullers and jacks are discussed in a new bulletin, Hydraulic 51, issued by Templeton, Kenly and Co. Simplex Re-Mo-Trol hydraulic pumps and rams are illustrated, as well as the "Center-Hole" tubular ram construction. The manufacturer states that no complicated fixtures are needed in prestressing operations as the ram furnishes its own back-up. Specifications are included.—Templeton, Kenly and Co., 1020 S. Central Ave., Chicago 44, Ill.

Air vibrators

The use of air vibrators in various phases of concrete products production is described in recent literature from Cleveland Vibrator Co. Different type vibrators are illustrated for use on hoppers bins and chutes, batch mixers, pipe forms, wooden forms, unloading hopper railroad cars, vibration tables and for use in manufacturing concrete block and precast units. —Cleveland Vibrator Co., 2884 Clinton Ave., Cleveland 13, Ohio.

Steam-cured blocks

High pressure steam and vacuum cured concrete blocks are discussed in a bulletin issued by Harter Marblecrete Stone Co., Inc. Test data is presented to illustrate the advantages of high pressure steam curing and specifications for concrete masonry are included.—Harter Marblecrete Stone Co., Inc., 1614 West Main Street, Oklahoma City, Okla.

Drilling concrete

A portable electric drill developed by Wodack Electric Tool Corp. can be used for three different types of operations: (1) drilling concrete and masonry, (2) drilling metal and wood and (3) driving various grinding, buffing and wire wheels, *i.e.*, as a hammer, drill and grinder. The manufacturer states that the Do-All drill can drill a ⅞ in. diameter hole in concrete at the rate of 3½ in. per minute.—Wodack Electric Tool Corp., 4627 W. Huron St., Chicago 44, Ill.

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 22 are currently available at prices indicated. Please order by title and title number.

SOME OBSERVATIONS ON THE USE OF REINFORCING STEEL IN CONCRETE PAVEMENTS.....47-1

Price 35 cents.

BENGT F. FRIBERG—Sept. 1950, pp. 1-16 (V. 47)

The paper contains a few accumulated observations from a review of literature on the evolution of reinforced concrete pavement designs, with reference to various systems of reinforcement which have seen extended use. Typical structural concrete pavement failures are described. Trends in design and reinforcement practice are shown, with special reference to recent developments. Pertinent findings of a few published pavement surveys are cited, with special attention to the 900-mile Louisiana pavement survey made in 1945. The need for performance information and additional research is stressed.

TESTS OF PAPER MOLDS FOR CONCRETE CYLINDERS.....47-2

Price 35 cents.

ROBERT A. BURMEISTER—Sept. 1950, pp. 17-24 (V. 47)

Concrete test cylinders cast in a new type paper mold had a compressive strength lower than that predicted for the concrete mix used. Investigation showed that cracks and mechanical injuries to the outer shell of the concrete cylinder caused by movement of the paper stock during the first 24 hours of curing reduced the strength of the cylinder. To a lesser degree this was true also of the paraffined paper molds in common use for casting test cylinders, specimens cast in both types of paper molds showing lower strengths than test cylinders from the same mix cast in steel molds.

ADMIXTURES IN CONCRETE.....47-3

Price 60 cents.

W. T. MORAN, F. H. JACKSON, BRUCE E. FOSTER and T. C. POWERS—Sept. 1950, pp. 25-52 (V. 47)

Five papers by members of ACI Committee 212, Admixtures, are, because of their common general subject, presented together.

Various admixtures are discussed briefly as an introduction to more detailed treatment of air-entraining materials. The relative merits of admixtures and interground agents are considered. Optimum ranges of air content for different structural uses are given with particular reference to pavements.

The advantages and disadvantages of several types of admixtures used in the fabrication of various concrete products, such as building block, cast stone, pipe, cribbing and curbing, are discussed. The admixtures considered are classified into the following groups: accelerators, air-entraining agents, gas-forming agents, water repellent agents, and workability agents.

The factors affecting bleeding characteristics and workability of fresh concrete are reviewed and the effect of admixtures on these properties is assessed.

Present knowledge of admixtures in counteracting alkali-aggregate reactions is reviewed. It is emphasized that further studies may revise thinking in this field.

The effectiveness of various concrete admixtures in inhibiting the capillary flow of water and the flow of water under pressure is considered. The types of admixtures included in the discussion are accelerators, soaps, butyl stearate, finely subdivided dry materials, mineral oil, workability agents, and a miscellaneous group of proprietary compounds.

IMPROVED SONIC APPARATUS FOR DETERMINING THE DYNAMIC MODULUS OF CONCRETE SPECIMENS.....47-4

Price 35 cents.

C. E. GOODELL—Sept. 1950, pp. 53-60 (V. 47)

After a brief introduction to sonic testing and a description of commercial equipment, the apparatus built for the Michigan State Highway Department is discussed. Reliable results can be obtained by an unskilled worker with this compact equipment which has twice the frequency band spread of the usual oscillator. A wiring diagram of the sonic apparatus is included.

ANALYSIS OF THREE-DIMENSIONAL BEAM-AND-GIRDER FRAMING.47-5

Price 35 cents.

PHIL M. FERGUSON—Sept. 1950, pp. 61-72 (V. 47)

The beam-and-girder floor with some beams carried directly by columns and others supported on girders is cited as a practical problem in frame analysis that must include the torsional stiffness of the girder. Curves showing how moment coefficients vary with this torsional stiffness are developed for a few simple cases in interior panels. These show the weakness of rule-of-thumb methods.

A practical calculation form is set up for use with the moment distribution method in solving three-dimensional problems of this type.

Proposed Revision of SPECIFICATIONS FOR CONCRETE PAVEMENTS AND BASES (617-44).....47-6

Price 35 cents.

REPORT OF COMMITTEE 617—Oct. 1950, pp. 93-116 (V. 47)

New specifications incorporated include those covering air entrainment, removal of forms, premolded joint fillers and joint filling materials and method of placing reinforcement. Definitions have been added under soil foundation preparation and other parts of this section have been revised.

LINEAR TRAVERSE TECHNIQUE FOR MEASUREMENT OF AIR IN HARDENED CONCRETE.....47-7

Price 35 cents.

L. S. BROWN and C. U. PIERSON—Oct. 1950, pp. 117-124 (V. 47)

The method described for the determination of air in hardened concrete permits the examination of 6 x 8-in. and 6 x 10-in. random plane face-ground hardened concrete specimens which more truly represent the aggregate and air voids in the actual concrete than smaller specimens. The construction and use of the instruments are discussed and results of tests are given. Because of the time and equipment necessary to measure air content by means of the integrator, it is not adaptable to field use. However, as a laboratory tool it provides a means for quick and accurate determination of total air.

INFLUENCE OF THE QUALITY OF MORTAR AND CONCRETE UPON CORROSION OF REINFORCEMENT.47-8

Price 35 cents.

RACHEL FRIEDLAND—Oct. 1950, pp. 125-140 (V. 47)

In tests to determine the influence of the quality of mortar and concrete upon corrosion of reinforcement the variables studied were cement content, water-cement ratio, consistency, grading and depth of cover. The specimens, stored in moist air or exposed to weather, were tested up to the age of 2 years.

The results indicate that consistency has a pronounced effect upon the protective value of mortar and concrete, and that there appears to exist an "optimum consistency" at which the quantity of rust is practically unaffected by time. It was also found that the usual cement contents in reinforced concrete have only a limited effect upon corrosion. It is concluded that water-cement ratio does not in itself control the rate of corrosion of reinforcement.

PROPER SAND GRADING IMPROVES MASS CONCRETE.....47-9

Price 35 cents.

THOMAS B. KENNEDY—Oct. 1950, pp. 141-152 (V. 47)

Two series of concrete mixtures were designed using 6-in. traprock coarse aggregate and eight separate gradings of natural sand ranging in fineness modulus from 3.60 to 1.35. Tests were made of the plastic concrete, and specimens were cast for tests of compressive strength, resistance to freezing and thawing, and drying shrinkage. Both series of concrete mixtures had a cement content of 2.5 bags per cu yd; one had a normal air content—4½ ± 1½ percent in the portion of the mixture passing the 1½-in. sieve—and the other had a high air content—10 ± 2 percent in the portion of the mixture passing the 1½-in. sieve. Tests indicate that good durability in freezing and thawing can be obtained within the normal air content range with fineness modulus between 2.50 and 2.90. With increased air content, however, the fineness modulus range can be increased to extend from 1.58 to 3.24. Compressive strength was generally affected adversely by increased air content, but not to a serious degree. Drying shrinkage was less with normal air content mixes than with high air content mixtures. It was least when a fineness modulus of 2.52 was used, little difference being apparent between the high and normal air content mixes with this fineness modulus. The air-entraining admixture requirement increased greatly as the fineness modulus of the sand decreased. The water ratio also tended to increase with decreasing fineness modulus.

WATER-SOLUBILITY OF ALKALIES IN PORTLAND CEMENT.....47-10

Price 35 cents.

J. L. GILLILAND and T. R. BARTLEY—Oct. 1950, pp. 153-160 (V. 47)

In an effort to show correlation of soluble alkalies with alkali-aggregate reaction, the authors hydrated a number of cements for periods up to 90 days and analyzed water extracts of the ground hydrated cement. However, the correlation with expansions of mortar bars prepared with reactive aggregate was not improved by considering water-soluble alkalies rather than total alkalies.

The rate at which the alkalies become water-soluble in hydrating cement indicates that the alkali-bearing phases in cement hydrate quite readily.

NEW PRESTRESSING METHOD UTILIZES VACUUM PROCESS.....47-11

Price 35 cents.

K. P. BILLNER—Oct. 1950, pp. 161-176 (V. 47)

The method outlined here was developed to simplify prestressing of concrete to make it generally adaptable to American ways of construction. It eliminates costly anchorages; uses large diameter wires (⅝-in. diameter now available on the market), instead of the customary ⅛-in. diameter wire, thus greatly reducing the number of wires required; prestresses all the wires in the building element simultaneously and simplifies forming. A simplified method of design calculations for prestressed concrete and the result of tests of a beam so designed are included.

PROPOSED RECOMMENDED PRACTICE FOR THE APPLICATION OF MORTAR BY PNEUMATIC PRESSURE 47-12

Price 35 cents.

REPORT OF COMMITTEE 805—Nov. 1950, pp. 185-196 (V. 47)

This proposed ACI Standard presents briefly the advantages and disadvantages of pneumatically-placed mortar and establishes recommended practices for placing and mixing shotcrete, qualifications and duties of workmen, preparation of surface before shotcreting, reinforcing, sequence of application, and other items involved in good shotcreting.

DETERMINING OPTIMUM CROSS SECTIONS FOR PRESTRESSED CONCRETE GIRDERS.....47-13

Price 35 cents.

FRED J. UZIEL—Nov. 1950, pp. 197-212 (V. 47)

General solutions are presented for selecting economically optimum cross sections for prestressed concrete flexural members. Such a solution for simply-supported girders of rectangular cross section, for any span and load, assuming the wires prestressed after setting of the concrete and full dead weight to act during the prestressing operation, leads to a design procedure which, in the case considered, is extremely simple and rapid to apply. It also permits comparisons and studies related to the critical cross section to be made in a more general and conclusive manner. For cross sections other than rectangular, the number of variables making the solution more indeterminate in nature, a similar direct solution is not obtainable. However, a rapid way of obtaining the minimum areas of concrete and steel required is suggested.

STEAM CURING PROTECTS WINTER CONCRETING.....47-14

Price 35 cents.

C. O. CRANE—Nov. 1950, pp. 213-216 (V. 47)

The use of live steam for protecting newly placed concrete from freezing weather and for providing initial curing has resulted in excellent concrete in the Enders Dam spillway at no greater cost than less desirable dry heating methods. This brief paper describes in detail the methods used.

ANALYSIS OF CONTINUOUS CIRCULAR CURVED BEAMS.....47-15

Price 35 cents.

BECLA VELUTINI—Nov. 1950, pp. 217-228 (V. 47)

Continuous circular curved beams can be analyzed easily by the moment distribution method if both bending and torsional end couples are considered. Formulas and tables are presented for circular curved beams of constant cross section that give the relations between the end moments and end torques and the corresponding rotations of the end sections. A proposed method of procedure is illustrated in which the bending end couples are kept separate from the torsional couples. The mathematical operations are not difficult as the convergence is rapid, but attention must be paid to the sign convention which must be definite and easy to apply.

SHEAR RESISTANCE OF TILE-CONCRETE FLOOR JOISTS.....47-16

Price 35 cents.

J. NEILS THOMPSON and PHIL M. FERGUSON—Nov. 1950, pp. 229-236 (V. 47)

Tests on certain types of tile-concrete joists indicate that the tile webs are more effective in resisting diagonal tension than is indicated by the current ACI Building Code specification. Stagger of tile joints appears to be unnecessary, since they do not seem to be planes of weakness insofar as diagonal tension is concerned. The tile reduces the deflection of the joist.

SOLUTION OF DIFFICULT STRUCTURAL PROBLEMS BY FINITE DIFFERENCES.....47-17

Price 35 cents.

ALFRED PARME—Nov. 1950, pp. 237-256 (V. 47)

Finite differences can be applied to the solution of those structural problems in which the physical relationships are expressed as a differential equation. Essentially, the technique employed consists of replacing the derivatives of the differential equation by its central difference equivalent. The problem is thus reduced to the simple task of solving a system of simultaneous linear algebraic equations. The numerical computation involved in the procedure is considerably reduced by two devices. First, the number of equations neccessary to attain sufficient accuracy is reduced by an evaluation of the error introduced in substituting central differences for derivatives. Secondly, the solution of simultaneous equations is speeded by a systematic rapid tabulation of easily determined values. The procedure is applied to the design of a sheet pile wall, elliptical dome and skewed bridge to illustrate the scope and simplicity of the method.

Proposed Revision of BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-47).....47-18

Price 35 cents.

REPORT OF COMMITTEE 318—Dec. 1950, pp. 269-276 (V. 47)

Proposed changes decrease the allowable bond stress in plain bars (including the old types of deformed bars) and increase the allowable bond stresses for the new types of bars over those previously allowed for the old types. Top bars, those having more than 12 in. of concrete under them, are assigned lower bond stresses than bars in other positions. All plain bars must be hooked, which corresponds to special anchorage under the old provisions. The new bars develop sufficient anchorage by bond alone to correspond to special achorage with the old type bars. Consequently, all bars under the new provisions correspond to those with special anchorage under the old provisions.

CURING CONCRETE PAVEMENTS WITH MEMBRANES.....47-19

Price 35 cents.

C. C. RHODES—Dec. 1950, pp. 277-296 (V. 47)

To provide data to assess the advisability of continuing membrane curing of concrete pavements as an alternate method, laboratory and field tests were made to compare the effect of storage conditions on the physical properties of concrete, warping, temperature control, and strength and abrasion resistance of concrete cured with membranes and with wet burlap. A survey of pavements cured with clear membranes in spring and summer showed that cracking, when it occurred at all, was found predominantly in pavements laid in the morning hours. Comparing white-pigmented membranes with the usual wet-curing conditions in the field it was found to be efficient, practicable and about half as expensiv s wet curing under the same conditions.

BLADE CHANGES IMPROVE TILTING MIXER.....47-20

Price 35 cents.

GLENWAY MAXON—Dec. 1950, pp. 297-300 (V. 47)

Recent experiments, as well as earlier studies, on changing the blading of tilting concrete mixers so as to improve the quality of the mixed concrete are described. The evolution of the blade shapes and the effect of these changes on the path of the materials through the mixer are illustrated.

PROTOTYPE PRESTRESSED BEAM JUSTIFIES WALNUT LANE BRIDGE DESIGN.....47-21

Price 35 cents.

GUSTAVE MAGNEL—Dec. 1950, pp. 301-316 (V. 47)

The tests made on a prestressed concrete beam of 154 ft 8 in. span, identical to the beams of the main span of the Walnut Lane Bridge in Philadelphia, more than justified the adoption of prestressed concrete for the bridge. The test methods and results are described. The test beam exhibited a safety factor against cracking of about 2, which would be far lower for a reinforced concrete beam. The factor of safety against complete failure was about the same as for reinforced concrete while the deflection was less. It is possible to use prestressed concrete for beams where structural steel can not be used due to excessive deflection. The tests proved that the Walnut Lane Bridge will have an exceptional degree of safety with less weight and greater durability than would be possible with reinforced concrete, as well as being considerably cheaper than the conventional solution.

FINISHING AND CURING: A KEY TO DURABLE CONCRETE SURFACES.....47-22

Price 35 cents.

MYRON A. SWAYZE—Dec. 1950, pp. 317-332 (V. 47)

After a comparison of past and present pavement curing and finishing techniques the significance of timing and character of finishing and the timing and mode of curing are discussed. Laboratory tests are cited to show the effect of time of finishing and curing on surface durability to freezing and thawing. It is recommended that all concrete exposed to frost contain entrained air, have a low water-cement ratio and be thoroughly compacted after placing. A finishing and curing procedure is suggested which is adapted to the ambient conditions and to the hydration needs of the cement.

ECONOMY THROUGH BETTER CONTROL OF REINFORCING STEEL.....47-23

Price 35 cents.

F. TESSITOR and P. ROSEWARNE—Dec. 1950, pp. 333-340 (V. 47)

Difficulties and troubles encountered through the use of reinforcement steels not in strict accordance with present-day specifications are discussed. Data are presented to show the possibility of alleviating the situation for engineers, producers and contractors. Simplification of materials requirements, liberalization of code requirements to permit hot bending of bars and permanent identification of grade of steel would, in large measure, permit designers to apply reinforcement to structures in a more effective manner.

Proposed Revision of MANUAL OF STANDARD PRACTICE FOR DETAILING REINFORCED CONCRETE STRUCTURES (ACI 315-48).....47-24

Price 35 cents.

REPORT OF COMMITTEE 315—Jan. 1951, pp. 349-352 (V. 47)

Changes are proposed in bar designations to conform to the numbered designation of the U. S. Department of Commerce and in all drawings to agree with new bond values and anchorage details for new-style deformed bars. Editorial changes in the text of the Standard are outlined.

COARSE-GROUND CEMENT MAKES MORE DURABLE CONCRETE.....47-25

Price 35 cents.

HAROLD W. BREWER and RICHARD W. BURROWS —Jan. 1951, pp. 353-360 (V. 47)

A test procedure for mortar ring specimens is described. Rings containing coarse-ground cement shrunk less and showed greater resistance to freezing and thawing and to outdoor exposure than those containing fine-ground cement. These laboratory tests indicate that coarse-ground cement produces more durable concrete han fine-ground cement.

SIMPLE EQUIPMENT ECONOMICALLY EXPLORES PRESTRESSING..47-26

Price 35 cents.

MARVIN L. MASS and JACK R. JANNEY—Jan. 1951, pp. 361-364 (V. 47)

A tubular grip utilizing type metal to hold the prestressing wires is described. Other equipment includes a framework against which the prestressing force is applied and adjustable formwork for experimental beams. The equipment is so simple that it can be easily and inexpensively built to permit study of prestressed reinforced concrete in the small laboratory.

LABORATORY TESTS OF SPACED AND TIED REINFORCING BARS....47-27

Price 35 cents.

WILLIAM T. WALKER—Jan. 1951, pp. 365-372 (V. 47)

Tests of beam and pull-out specimens containing spaced and tied reinforcing arrangements indicated that little or no advantage would be obtained by spacing deformed

reinforcing bars at splices. Pull-out tests showed that for deformed bars, placed vertically, in which interlocking of lugs could take place there was a slight increase in strength due to tying the bars. Beam tests showed no significant difference between spacing and tying.

Twelve beams containing three different types of deformed bars either spaced or tied and extending the entire length of the specimens were tested. The sixty vertical pull-outs tested also contained three different types of deformed bars. Half of these were made with the bars spaced and half with the bars tied together. The reinforcing arrangement brought only one bar out of the specimens for pulling; the other was allowed to bear on the base plate.

Spacing of the bars in both beams and pull-outs conformed to the joint code minimum spacing requirement of 2½ diameters center to center with a minimum clear distance of 1½ times the maximum size of coarse aggregate.

FINISHING AIR-ENTRAINING CONCRETE PAVEMENTS.....47-28

Price 35 cents.

CHARLES W. ALLEN—Jan. 1951, pp. 373-376 (V. 47)

Based on a survey of state highway department practices, common difficulties in finishing air-entraining concrete pavements are enumerated. Practices in adjusting transverse finishing machines are given and revised finishing practices are discussed. Delayed finishing, as specified in several states, is considered in connection with the riding qualities of the finished surfaces. On the basis of experience in Ohio it is concluded that smooth pavements can be built of air-entraining concrete without delayed finishing.

EFFECTS OF LATERAL LOADS ON ARCHES.....47-29

Price 35 cents.

JAMES P. MICHALOS—Jan. 1951, pp. 377-388 (V. 47)

The effects of lateral loads on arches and the possible magnitude of these effects are considered. An influence table and influence lines are presented for moments and shears in unbraced parabolic arch ribs of constant cross section. These values are for several ratios of rise to span and several ratios of bending to torsional stiffness. The effect of haunching is studied and its possible importance is assessed. Procedures are presented for drawing approximate and exact curves of moments for unbraced arch ribs and for arch ribs braced with struts normal to the ribs. Numerical studies are included.

GETTING MORE FOR OUR CONCRETE DOLLAR.....47-30

Price 35 cents.

I. E. MORRIS—Jan. 1951, pp. 389-396 (V. 47)

A concrete slab with alternating horizontal elements connected by sloping elements is described and its design is worked out. The system is intended primarily for roofs where an exposed ceiling is desired for economy. It may be used to span considerable distances, either as a simple span or as a series of continuous spans. The members are designed to use both concrete and steel to maximum advantage.

FACTORS INFLUENCING CONCRETE STRENGTH.....47-31

Price 35 cents.

WALTER H. PRICE—Feb. 1951, pp. 417-432 (V. 47)

The effect of mix proportions, type and brand of cement, availability of moisture for curing, accelerators and curing temperatures on the rate and potential strength development of concrete are discussed. The influence of rate and frequency of load applications, dimensions of test specimens and lateral restraint on the indicated strength are also discussed, and information is furnished on the variations in strength which might be expected on a typical job. Compressive, tensile, flexural, bond and shearing strengths are compared, and the strengths of control cylinders are compared with the strengths of cores drilled from structures at later ages. Information is also furnished on strength loss from freezing and thawing and alkali-aggregate expansion.

SONISCOPE TESTS CONCRETE STRUCTURES.....47-32

Price 35 cents.

E. A. WHITEHURST—Feb. 1951, pp. 433-444 (V. 47)

The Soniscope, an instrument which measures group velocities through as much as 50 ft of concrete, was used for the field testing of 13 bridges, one navigation lock, 14 dams and five highway pavements in 12 states. Repeated tests permitted study of changes in the condition of the concrete and the development of group velocities indicating the condition of the structure. The value of the results increases with knowledge of the materials, mix design, method of placement and other characteristics of the structure being tested.

EFFECTIVE SEALING OF CONCRETE PAVEMENT JOINTS.....47-33

Price 35 cents.

H. F. CLEMMER—Feb. 1951, pp. 445-448 (V. 47)

The development of cork, metal and rubber joint sealers is traced and work with thermoplastic and cold-pour joint filling compounds is described. The importance of proper technique of preparing and placing the newer materials is emphasized.

ENTRAINED AIR SIMPLIFIES WINTER CURING.....47-34

Price 35 cents.

JOSEPH J. SHIDELER, HAROLD W. BREWER and WILBUR H. CHAMBERLIN—Feb. 1951, pp. 449-460 (V. 47)

The investigation described was undertaken to determine the winter protection required to protect air-entraining concrete from damage by freezing. Cylinders made with Types II and V cement and various percentages of calcium chloride were cured at temperatures ranging from 10 to 70 F and tested for strength at ages ranging from 3 to 180 days. The resistance of these concretes to accelerated freezing and thawing was compared. The results indicate that the amount of winter protection, as presently specified by the Bureau of Reclamation, can be reduced when air-entraining concrete is used.

PRECAST CONCRETE CONSTRUCTION IN CANADA.....47-35

Price 35 cents.

OTTO SAFIR—Feb. 1951, pp. 461-468 (V. 47)

Precast concrete construction techniques as applied to a warehouse, parking garage and retaining wall project are described. Beams and columns for the precast concrete frames of a warehouse were cast flat in multiple forms on the ground slab. Cold weather required special precautions in mixing, placing and curing the concrete. In a parking garage, precast elements were combined with cast-in-place members to form an essentially monolithic structure. A retaining wall 25-ft high is of precast counterforts and 8-in. wall planks. Principal features of the job were the complicated shape of the counterforts and the weight of the completed units.

FACTORS IN PRESTRESSED GIRDER DESIGN.....47-36

Price 35 cents.

M. FORNEROD—Feb. 1951, pp. 469-480 (V. 47)

Following a general description of the Walnut Lane Bridge the stages of loading of the prestressed concrete girders and the sequence of construction operations as they affected the design are discussed. The owner's stress limitations are listed and longitudinal bending in girders, transverse bending in stiffener diaphrams and shear and principal tensile stress in the girder are considered. Safety factors are considered for various loading conditions.

PRECAST CONCRETE OFFERS PROTECTION AGAINST ATOMIC BLAST.....47-37

Price 50 cents.

ARSHAM AMIRIKIAN—Mar. 1951, pp. 497-516 (V. 47)

Precast concrete is an excellent means for providing protection against atomic blast. The technique is suitable for new construction and is adaptable to existing structures by providing them with a protective shell. Readily-assembled framing elements can be prefabricated at regional plants and stored or stockpiled at various points for immediate use in an emergency.

For design purposes, in this paper, shelters are divided into three groups in relation to their proximity to a probable target. Suggested arrangements and main details of assembly of a number of types of precast framing suitable for each group are presented. A general discussion of the protection problem, the needed weapons data and design criteria are also given.

FALSE SET IN PORTLAND CEMENT.....47-38

Price 35 cents.

R. F. BLANKS and J. L. GILLILAND—Mar. 1951, pp. 517-532 (V. 47)

False set of cement causes difficulties in mixing and placing and even though the stiffening is eliminated by job conditions, extra mixing or the addition of corrective admixtures, undesirable effects on the hardened concrete remain. It adversely affects water requirement, strength, bond between aggregate and matrix, brittleness and cracking, resistance to freezing and thawing, and air-entraining characteristics.

Corrections in the manufacture of the cement are proper cooling of mills or the use of stable calcium sulfate.

DIRECT DESIGN OF T-BEAMS.....47-39

Price 35 cents.

HENRY J. COWAN—Mar. 1951, pp. 533-544 (V. 47)

In designing T-beams by usual methods some initial dimensions of the section must be assumed, either the complete dimensions—and the maximum concrete and steel stresses computed to ensure they are less than the allowable stress—or the effective depth only assumed—with tension area computed and the maximum compressive stress in the flange checked to find if compressive reinforcement is required. The author emphasizes that both methods are likely to produce uneconomical sections and take more time.

The direct design procedure proposed enables the dimensions of a T-beam to be calculated for any given set of conditions. Curves are set up for finding the value of j, and the solution of problems by normal methods and "direct design" are compared by solving six examples. An "equivalent flange" method is proposed for the balanced design of T-beams when the area of compression reinforcement is specified, and results show little difference between exact and approximate methods of design.

ALKALI-AGGREGATE EXPANSION CORRECTED WITH PORTLAND-SLAG CEMENT.....47-40

Price 35 cents.

FEDERICO BARONA DE LA O—Mar. 1951, pp. 545-552 (V. 47)

Rather than a pozzolan, granulated basic blast furnace slag described in ASTM Specification C205 should be considered a latent or potential cement with a high siliceous glass content that requires the presence of hydrated lime and gypsum to hydrate properly. To correct alkali-aggregate expansion, a high proportion of slag (about 50 to 60 percent) should be used, which does not reduce strength; rather than lower percentages (20 to 30) which can not be exceeded with pozzolans unless strength is sacrificed. The portland-blast furnace slag cement produced in Mexico, meets ASTM requirements, presents satisfactory strength, low heat of hydration and low alkali content. The corrective action was investigated using Pyrex glass as reactive aggregate, and *NaOH* to increase the alkali content of the different cements and blends to the same high value (1.23 percent).

Under similar conditions, with the same high alkali content, much smaller expansions were obtained with slag blends than with straight portlands. It is not intended to compare or recommend the use of slag instead of pozzolans, or the use of portland-blast furnace slag cement instead of modified, low-heat, or sulfate resistant portland cement, since in each case, the availability, cost and special conditions will determine what to use. In many cases, slag can be used advantageously to replace 50 to 60 percent of portland clinker.

LEAN MASS CONCRETE USED FOR INTERIOR OF DAMS.....47-41

Price 35 cents.

BYRAM W. STEELE—Mar. 1951, pp. 553-560 (V. 47)

It is not necessary for concrete in the interior and exterior of a dam to have equal durability. By placing 4-bag concrete on exterior faces and lean mass concrete (2½ bag mix) in the interior, it is possible to achieve both durability and economy in gravity dam construction. Lean mass concrete is important in minimizing volume-change cracking; the ensuing economy is an additional feature. The use of interior and exterior mixes in Corps of Engineers dams is described.

Membership certificates

Repeated but infrequent requests for membership certificates indicate that enough members may want them to warrant having them made. At its fall meeting, the Board of Direction authorized the Secretary to determine through this notice whether or not the Institute would be justified in going to the expense of having the necessary work done.

It is estimated that certificates suitable for framing can be furnished ACI Members at a cost of not over $3.50. Those interested are requested to clip the attached coupon and mail it to Institute headquarters. *Please do not send money until billed.* If there are not enough interested, the Institute would not be justified in supplying certificates to a few at a loss.

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DISCUSSION

Discussion closed January 1, 1951

Sept. Jl. '50

Some Observations on the Use of Reinforcing Steel in Concrete Pavements—Bengt F. Friberg

Tests of Paper Molds for Concrete Cylinders—Robert A. Burmeister

Admixtures in Concrete—W. T. Moran, F. H. Jackson, Bruce E. Foster and T. C. Powers

Improved Sonic Apparatus for Determining the Dynamic Modulus of Concrete Specimens—C. E. Goodell

Analysis of Three-Dimensional Beam-and-Girder Framing—Phil M. Ferguson

Discussion closed February 1, 1951

Oct. Jl. '50

Proposed Revision of Specifications for Concrete Pavements and Bases—Committee 617

Linear Traverse Technique for Measurement of Air in Hardened Concrete—L. S. Brown and C. U. Pierson

Influence of the Quality of Mortar and Concrete upon Corrosion of Reinforcement—Rachel Friedland

Proper Sand Grading Improves Mass Concrete—Thomas B. Kennedy

Water-Solubility of Alkalies in Portland Cement—J. L. Gilliland and T. R. Bartley

New Prestressing Method Utilizes Vacuum Process—K. P. Billner

Discussion closed March 1, 1951

Nov. Jl. '50

Proposed Recommended Practice for the Application of Mortar by Pneumatic Pressure—Committee 805

Determining Optimum Cross Sections for Prestressed Concrete Girders—Fred Uziel

Steam Curing Protects Winter Concreting—C. O. Crane

Analysis of Continuous Circular Curved Beams—Becla Velutini

Shear Resistance of Tile-Concrete Floor Joists—J. Neils Thompson and Phil M. Ferguson

Solution of Difficult Structural Problems by Finite Differences—Alfred Parme

How well is ACI serving the cement and concrete industry? Three major phases of Institute activity are examined—annual and regional meetings, publications, committee activities.

ACI's Place in a Billion Dollar Industry*

By FRANK H. JACKSON†

SYNOPSIS

Retiring ACI President Jackson examines Institute activities in relation to the concrete construction industry. ACI has won for itself a high place among the national professional engineering societies, but that eminence cannot be held without continual vigilance and increased effort on the part of all Institute members. Three major phases of activity are examined—annual and regional meetings, publications and committee functions. Recommendations are presented for improving and increasing Institute activities.

FROM FOUNDATIONS TO FURNITURE

The production of portland cement in the United States reached an all time high last year of 225,000,000 barrels‡—enough cement to build 20 Grand Coulee dams or a four-lane concrete highway around the world at the equator. The actual value of all concrete construction during 1950 would be difficult to determine. However, if we assume that 90 percent of this cement was used in concrete at an average rate of one barrel per cubic yard and that the concrete had an average value of $10 per yd, we arrive at the impressive figure of roughly two billion dollars for the cost of the concrete alone. Add to this design and construction costs and we have accounted for a substantial part of the 28 billion dollars that has been estimated as the total cost of all construction activities in this country during 1950.

These figures are cited to illustrate that the American Concrete Institute is indeed associated with a billion dollar industry, an industry which has grown tremendously in size and importance during a period which corresponds almost exactly with the life of our organization. Many applications, particularly in the field of reinforced concrete, which are now commonplace were undreamed of at the turn of the century when the uses of concrete were confined almost entirely to mass and foundation work, sidewalks and concrete block. Compare this to the hundreds of ways in which we are

*Presented at the ACI 47th Annual Convention, San Francisco, Calif., Feb. 21, 1951. Title No. 47-42 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 8, Apr. 1951, *Proceedings* V. 47. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Aug. 1, 1951. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Principal Engineer of Tests, U. S. Bureau of Public Roads, Washington, D. C.

‡U. S. Bureau of Mines.

now using concrete, such as industrial, commercial and business buildings, housing, pavements for highways and airports, highway and railway bridges, even bridges that float, trestles, viaducts and tunnels, chimneys, farm structures, hydraulic and water impounding structures, flood control works, and an almost endless variety of precast products including building units of all sizes and types, ranging from ordinary building block to full size wall and roof sections and precast structural elements, sewer and water pipe, drain tile, light and telephone poles, fence posts, cement-asbestos products, lightweight concrete units and garden furniture. We have even built concrete ships and there is no reason why we should not do so again when and if it can be justified economically. Truly this is an amazing list.

The rapid growth and present pre-eminence of concrete in the construction field is no accident. The possibility of combining concrete and steel to eliminate concrete's one structural weakness, low tensile strength, combined with its potentialities for architectural treatment make it an ideal material for building construction. Its excellent resistance to hurricane, flood, earthquake and bomb damage make it ideal for construction work in areas subjected to such disturbances. Its value from the standpoint of both fire-resistance and sanitation has made it exceedingly attractive to the farmer. And finally, its inherent durability, or weather resistance, made even greater by the introduction of air entrainment, has resulted in its extensive use in exposed structures of every type.

RECENT DEVELOPMENTS

These are rapidly changing times in every phase of human endeavor, and the art of doing concrete work is no exception. Three recent developments are highly significant. These are (1) the increasing use of admixtures in concrete, (2) new developments in precast structural concrete and (3) developments in the application of prestressing.

It is only recently that the word "admixture" has attained the status of respectability so far as concrete usage is concerned. For years we were told that the only ingredients necessary in making concrete were portland cement, aggregates and water. These were the basic ingredients, and the use of any additional material was frowned upon. Then came air entrainment and we soon found that even good concrete could be made better by the use of an admixture. We have learned also that many of the properties of concrete can be improved by replacing substantial amounts of portland cement with a natural or artificial pozzolanic material. These are being used more and more in various types of work either added separately or interground with portland to make blended cements. The possibility of utilizing the large quantities of fly ash that are available in many localities is only one of many aspects of this development.

The increasing use of precast structural members in place of the conventional cast-in-place monolithic type of construction is also of great interest.

The need for low cost housing has created a demand for economical methods of mass construction that is being met by precasting the various structural elements. The obvious advantages of precast construction include elimination of on-the-site formwork (an important item in these days of high costs), close manufacturing control and the possibilities of economical design as compared to conventional procedures.

It is only recently that American designers have awakened to the possibilities of prestressing. Although the process was invented by an American and we pride ourselves on our progressiveness, we have lagged far behind European engineers in developing and applying the theory to actual construction. In Europe, prestressing has passed the experimental stage and is firmly established, whereas in this country the construction of the Walnut Lane Bridge in Philadelphia represents our first attempt to apply the theory to a major structure. However, interest seems to be increasing rapidly. Considerable research is under way and a national conference, in which ACI is cooperating, is being planned to explore the whole problem with special reference to engineering and economic phases peculiar to this country.

ACI ACTIVITIES

How does the American Concrete Institute fit into this picture? How are we now serving and how can we improve our service to meet the need of this important and growing industry?

The Institute was organized in 1905 as the National Association of Cement Users. The original purpose was to bring some order out of the chaotic state of the cement block industry at that time. Its first president, Richard L. Humphrey, served until 1914 and through his efforts and those of such pioneers as Leonard C. Wason, Henry C. Turner, Maxwell M. Upson and Alfred E. Lindau, to say nothing of Harvey Whipple, Secretary-Treasurer since 1919, the status of our organization was raised from what really amounted to a restricted trade group to that of a truly national society dedicated to nonprofit service in the field of concrete. In recognition of this broader objective, the name was changed to American Concrete Institute in 1913 and it was to emphasize still further this concept of service that the Institute was reincorporated in 1945 with the following statement of objectives:

> ". . . . to organize the efforts of its members for a nonprofit public service in gathering, correlating and disseminating information for the improvement of the design, construction, manufacture, use and maintenance of concrete products and structures. To this field of engineering education its resources shall forever be devoted."

By and large, the Institute has met the obligations for service outlined in its charter. It has done a good job and has won a high place among the national professional engineering societies of this country. A glance at the ACI 20-YEAR INDEX will show the extent and the variety of information that has been made available in the JOURNAL through the volunteer efforts of a comparatively few men who were willing and able to put the results of

their research and experience on paper for the benefit of their fellow workers. I for one do not apologize for our showing. It has been good.

This does not mean that we can afford to be complacent about our record. A self-satisfied, complacent attitude on the part of any organization is fatal. We should continually scrutinize our methods and ask ourselves whether changes in emphasis or in the direction of our efforts may not be desirable.

The three major phases of Institute activity are annual and regional meetings, publications, of which the major item is the JOURNAL, and committee activities.

Annual conventions

The annual conventions have in general been successful. Technical programs are almost invariably of high caliber and cover a wide variety of interests within our field. Although concurrent meetings are not desirable, they are necessary due to the large number of worthwhile papers available for presentation. The research session, started several years ago to give an opportunity for off-the-record presentations and discussion of current material, has become a fixture. I have no criticism of the annual meeting. My only regret is that the vast majority of our members are unable to attend them and thereby participate in the good fellowship and comradeship for men of kindred interests which these meetings afford.

Regional meetings

The first regional meeting was held in Birmingham, Ala., in 1948. It was highly successful due largely to the efforts of a group of local ACI members. The same can be said for the meeting the following year in Boston and the one last year in Washington. These meetings make it possible for many members and friends of ACI to get together who could not attend the regular annual convention. They have proved their value and are a distinct must for the future.

Publications

The JOURNAL is probably ACI's most important single activity. This is the medium through which the product of the research activities and the experiences of contributors reach those who can profit from their work. Furthermore, it is the only way we can reach and serve the bulk of our membership. JOURNAL content is therefore of the highest importance, not only from the standpoint of technical excellence, which goes without saying, but also the necessity of securing the widest possible reader interest. This means diversity of subject matter which in turn means either shorter technical papers and more of them or some other outlet through which the substance of our vast storehouse of knowledge of concrete can be made available in usable form to 5000 present and many additional thousands of potential members.

This question comes up periodically for discussion in the Board of Direction and there have been, for some time, two schools of thought as to the Institute's obligations. One group believes that the pages of the JOURNAL should be reserved for papers and reports of high technical value, including research

reports and that such reports should be complete with all substantiating data. Another group feels that papers and reports should be brief and concise and that research reports in particular should be limited to a condensed statement of objectives, procedures and conclusions, with supporting data available in mimeograph or similar form for the information of the comparatively few members who want such data.

This group believes that only in this way will it be possible for the Institute with its limited resources to give maximum service in terms of the things most wanted by the most people. They believe that only by broadening our publication policy to include more "how-to-do-it" articles will we be in a position to attract a substantial number of new dues-paying members and thereby provide the necessary funds for an expanding service to the industry. The chief difficulty lies in the matter of authorship. Men in the construction field, men who do things and who have developed valuable techniques as the result of their doing are not, as a rule, natural writers. They do not have a "yen" for that sort of thing and, as Roy W. Crum once said, a man must have a yen to write to produce a worthwhile paper. Also, we must remember that everything we produce in the Institute is the result of volunteer effort. We have no staff writers and under ACI's present budget limitations we have no funds to employ them. It is a knotty problem and I can see no immediate solution. The Board of Direction is giving the matter serious consideration and everything possible will be done to expand Institute services to the utmost.

Although we have no staff writers in the usual sense of the term we do have a small but capable editorial staff. This has resulted in a more careful and thorough editing of papers than was formerly possible. Editorially as well as technically the JOURNAL ranks high among engineering periodicals.

In addition to the JOURNAL, we have published a number of standards—mostly in the form of recommended procedures. Outstanding among these are the "Building Code Requirements for Reinforced Concrete" and the "Manual of Standard Practice for Detailing Reinforced Concrete Structures." Continued effort is being made to keep these and other standards abreast of the times. Both the code and the manual are being revised this year to take advantage of improvements in design made possible by the development of the high bond reinforcing bar.

The development of such standards as the Building Code, which is used as the standard for reinforced concrete design in the building codes of most of our large cities and the Detailing Manual, used extensively in the office, drafting room and school, rank among the most important ACI activities and illustrate very well some of the ways that our organization can be of service to the industry. We should have more standards of this type.

Technical committee activities

If there is one phase of Institute endeavor that has not functioned as well as it should, it is our technical committee activities. I am not criticising any

particular committee nor am I questioning the value and high caliber of the work done by the various committees. I am concerned entirely with rate of progress. This does not mean of course that all of our committees are inactive. Some of them I happen to know have been very active. Nevertheless, committees of the Institute just like committees of similar organizations have a tendency to stagnate unless they have a specific and clearly defined objective together with an urge, spark-plugged by someone, usually the chairmen, to get the job done. I am afraid that this is the situation so far as some of our committees are concerned.

In an attempt to overcome this tendency, the Technical Activities Committee recently recommended to the Board that the rules governing committee activities be revised to require that members be appointed for a two-year term and that all reappointments be contingent on continued activity in committee work. This should make it easier to eliminate "dead wood" and introduce new blood into committee work. Speaking of new blood, I wonder if you realize that out of a membership of over 5000, there are just 152 individuals or 3 percent who are members of technical committees. Fifty of these members belong to two or more committees and 17 of them belong to three or more committees. I wonder if the time has not come to give some of our younger men an opportunity to participate in committee work.

At the present time our 30 technical committees (excluding Committee 115, Research) average only 8 members each. I believe that many of them could be enlarged by the addition of some of the younger men. It is inconceivable to me that of approximately 5000 members not now affiliated with any committee, there are not some who would make excellent committee material.

There is just one more phase of Institute activity to which I should like to refer briefly—field control. It is a field which has been sadly neglected. Nevertheless, it is an obligation equally as important as the task of analyzing and correlating the findings of research or the writing of specifications. Although we have made considerable progress in closing what Sheets many years ago so aptly called the gap between theory and practice we still have far to go. Let me cite just one example. It is a matter of common knowledge among concrete men that, in many places, field procedures for control of the quality of ready-mix concrete are quite inadequate, if not actually nonexistent. Although we who are in the business of building highways pride ourselves on the quality of our field inspection in general, the control of ready-mixed concrete is a major problem of the materials departments of many states. It is even more of a problem with some municipalities.

Ready-mixed concrete constitutes an important and rapidly expanding branch of the industry. In 1949, nearly 28 million cu yd were produced. It is here to stay and steps must be taken to develop adequate field control procedures. The industry itself is alive to this problem and has developed a recommended practice primarily for the use of its own operators. What is needed is a national standard sponsored by some such organization as

ACI that can be used as a code of practice by municipalities and other agencies concerned with safeguarding the quality of concrete in their structures. I believe the development of such a recommended practice is a responsibility of the Institute. We are not concerned here with a specification; ASTM has written a specification. We are concerned with procedures for enforcing the specification.

ACI FINANCES

Brief reference to our present financial situation is desirable. As you know, funds for the various services which ACI provides are derived mainly from three sources—member dues, sale of publications and advertising. Institute membership is now the highest in its history—5243 as of Jan. 31, 1951. Total net gain for 1950 was 331 or approximately 7 percent as compared to 11 percent in 1949 and 17 percent in 1948. Although the rate of gain appears to be decreasing, the membership picture in general is good. Publications sales were approximately the same as for the preceding two years. Receipts from advertising were about the same as in 1949 but considerably below the banner year of 1948. Total receipts for the year were $128,919.85, about the same as for the past three years but more than four times what they were in 1940.

Expenses during the year have also been the highest in our history, due principally to skyrocketing printing costs and other items essential to the continuing operation of the Institute. Net operating expenses for the year amounted to $137,659.14, leaving a deficit of $8739.29. This has decreased our working surplus to $27,652.69 compared to $36,391.98 in 1949 and $41,010.93 in 1948. These figures are quite disturbing. No organization can continue to draw upon its surplus to finance an operating deficit and survive. We are in the same situation as many similar organizations. Rapidly mounting costs have far outstripped receipts, which have remained at approximately the same level during the recent period of rapid inflation. Two courses of action were open to the Board. It could either order a drastic curtailment of operating costs which would mean reducing by a substantial amount the sum total of the services which you now receive and are entitled to receive or it could attempt some adjustment in revenue potential with the hope of increasing total receipts sufficiently to balance the budget. As you know, the Board took the latter course and the convention ratified the recommendation that members dues be increased. Needless to say, the Board was extremely reluctant to take this action. It felt, however, that this was the only way in which the Institute could continue to function without loss in service to members and that the membership at large would understand the situation and would continue its loyal support of our work as in past years.

CONCLUSION

We are gathered here in San Francisco for our 47th Annual Convention during a period of great national emergency. We know from experience

that the national defense program now getting under way will profoundly affect the economic life of the nation. We are in for another era of wartime scarcities and no one can foresee the end. All we know is that things will probably get worse before they get any better. The concrete industry along with others will be faced with a shortage of critical materials—cement, aggregates and reinforcing steel. The civilian consumption of concrete will be cut as nondefense building operations are curtailed but this cut will probably be offset by increased demands of the defense agencies. Therefore, in the allocation of critical materials, consideration will have to be given to the part played by concrete in our mobilization for defense so that the industry may do its part.

The American Concrete Institute as an organization stands ready to do its part. Our plans for the future are being made in the hope that we shall be able to carry on in our usual manner. However, if sacrifices such as the temporary curtailment of some of our normal functions are necessary they will be made cheerfully. They were necessary during the last war and they may be required again. Of one thing, however, I am confident. The Institute stands ready to cooperate in every way possible in the united effort to create an impregnable defense against any attempt to control our way of life, no matter how powerful it may be or in what form it may appear.

Title No. 47-43

ACI Standard

Building Code Requirements for Reinforced Concrete (ACI 318-51)*

Reported by ACI Committee 318

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SYNOPSIS

This code covers the proper design and construction of buildings of reinforced concrete. It is written in such a form that it may be incorporated verbatim or adopted by reference in a general building code, and earlier editions of it have been widely used in this manner.

Among the subjects covered are: Quality of concrete; allowable stresses; mixing, placing, curing and cold weather protection of concrete; forms; cleaning, bending, placing, splicing and protection of reinforcement; construction joints; general design considerations; flexural computations; shear and diagonal tension; bond and anchorage; flat slabs; columns and walls; and footings. The quality and testing of materials used in the construction are covered by references to the appropriate A.S.T.M. standard specifications.

*Adopted as a Standard of the American Concrete Institute at its 47th Annual Convention, Feb. 20, 1951 as reported by Committee 318; ratified by Letter Ballot Apr. 5, 1951. ACI 318-51 supersedes ACI 318-47 published Sept. 1947.

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†Appointed chairman by the ACI Board of Direction, Oct. 23, 1950, succeeding Chester L. Post, deceased.

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CHAPTER 1—GENERAL

101—Scope

(*a*) This code covers the use of reinforced concrete and plain concrete in any structure to be erected under the provisions of the general building code of which it forms a part. It is intended to supplement the provisions of the general code in order to provide for the proper design and construction of structures of these materials. In all matters pertaining to design and construction where this code is in conflict with other provisions of the general code, this code shall govern.

102—Permits and drawings

(*a*) Drawings and typical details of all reinforced concrete construction showing the size and position of all structural members, metal reinforcement, design strength of concrete, and the live load used in the design shall be filed with the building department as a permanent record before a permit to construct such work will be issued. All plans submitted for approval or use on the work shall clearly show the strength of concrete at a specified age for which all parts of the structure were designed. Calculations pertaining to the design shall be filed with the drawings when required by the Commissioner of Buildings.

103—Special systems of reinforced concrete

(*a*) The sponsors of any system of reinforced concrete which has been in successful use, or the adequacy of which has been shown by test, and the design of which is either in conflict with, or not covered by this code shall have the right to present the data on which their design is based to a "Board of Examiners for Special Construction" appointed by the Commissioner of Buildings. This Board shall be composed of competent engineers, architects and builders, and shall have the authority to investigate the data so submitted and to formulate rules governing the design and construction of such systems. These rules when approved by the Commissioner of Buildings shall be of the same force and effect as the provisions of this code.

104—Definitions

(*a*) The following terms are defined for use in this code:

Aggregate, fine—Natural sand, or sand prepared from stone, blast furnace slag or gravel, or, subject to the approval of the Commissioner of Buildings, other inert materials having similar characteristics.

Aggregate, coarse—Crushed stone, gravel, blast furnace slag, or other approved inert materials of similar characteristics, or combinations thereof having hard, strong, durable pieces, free from adherent coatings.

Column—An upright compression member the length of which exceeds three times its least lateral dimension.

Column capital—An enlargement of the end of a reinforced concrete column designed and built to act as a unit with the column and flat slab.

Column strip—A portion of a flat slab panel one-half panel in width consisting of the two adjacent quarter-panels on either side of the column center lines and extending through the panel in the direction of the span considered for bending.

Combination column—A column in which a structural steel member, designed to carry the principal part of the load, is wrapped with wire and encased in concrete of such quality that some additional load may be allowed thereon.

Composite column—A column in which a steel or cast-iron structural member is completely encased in concrete containing spiral and longitudinal reinforcement.

Concrete—A mixture of portland cement, fine aggregate, coarse aggregate and water.

Deformed bar—A reinforcing bar conforming to the "Standard Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement" (ASTM Designation: A 305). Bars not conforming to these specifications shall be classed as plain bars. Wire mesh with welded intersections not farther apart than 6 in. in the direction of the principal reinforcement and with cross wires not smaller than No. 10 W. & M. gage may be rated as a deformed bar.

Drop panel—The structural portion of a flat slab which is thickened in the area surrounding the column or column capital.

Effective area of concrete—The area of a section which lies between the centroid of the tensile reinforcement and the compression face of the flexural member.

Effective area of reinforcement—The area obtained by multiplying the right cross-sectional area of the reinforcement by the cosine of the angle between its direction and the direction for which the effectiveness is to be determined.

Flat slab—A concrete slab reinforced in two or more directions, generally without beams or girders to transfer the loads to supporting columns.

Middle strip—A portion of a flat slab panel one-half panel in width, symmetrical about the panel center line and extending through the panel in the direction of the span considered for bending.

Panel length—The distance along a panel side from center to center of columns of a flat slab.

Pedestal—An upright compression member whose height does not exceed three times its least lateral dimension.

Plain concrete—Concrete without reinforcement, or reinforced only for shrinkage or temperature changes.

Reinforced concrete—Concrete in which reinforcement other than that provided for shrinkage or temperature changes is embedded in such a manner that the two materials act together in resisting forces.

Surface water—The water carried by the aggregate except that held by absorption within the aggregate particles themselves.

105—A.S.T.M. Specifications cited in this code*

The specifications of the American Society for Testing Materials referred to in this code are listed below with their serial designation including the year of latest revision. They are declared to be a part of this code the same as if fully set forth elsewhere herein:

A7-49T Tentative Specifications for Steel for Bridges and Buildings
A15-50T Tentative Specifications for Billet-Steel Bars for Concrete Reinforcement
A16-50T Tentative Specifications for Rail-Steel Bars for Concrete Reinforcement
A44-41 Standard Specifications for Cast Iron Pit-Cast Pipe for Water or Other Liquids
A82-34 Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement
A160-50T Tentative Specifications for Axle-Steel Bars for Concrete Reinforcement
A185-37 Standard Specifications for Welded Steel Wire Fabric for Concrete Reinforcement
A305-50T Tentative Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement.
C31-49 Standard Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Field
C33-49 Standard Specifications for Concrete Aggregates
C39-49 Standard Method of Test for Compressive Strength of Molded Concrete Cylinders
C42-49 Standard Methods of Securing, Preparing and Testing Specimens from Hardened Concrete for Compressive and Flexural Strengths
C94-48 Standard Specifications for Ready-Mixed Concrete
C130-42 Standard Specifications for Lightweight Aggregates for Concrete
C150-49 Standard Specifications for Portland Cement
C192-49 Standard Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Laboratory

*The A.S.T.M. specifications listed were the latest editions at the time this code was prepared. Since these specifications are frequently revised, generally in minor details only, the user of this code should check directly with the A.S.T.M. if it is desired to refer to the latest editions.

CHAPTER 2—MATERIALS AND TESTS

200—Notation

D = Deflection of a floor member under load test relative to the ends of the span L.

L = Span of member under load test (the shorter span of flat slabs and of floors supported on four sides).

t = The total thickness or depth of a member under load test.

201—Tests

(*a*) The Commissioner of Buildings, or his authorized representative, shall have the right to order the test of any material entering into concrete or reinforced concrete to determine its suitability for the purpose; to order reasonable tests of the concrete from time to time to determine whether the materials and methods in use are such as to produce concrete of the necessary quality; and to order the test under load of any portion of a completed structure, when conditions have been such as to leave doubt as to the adequacy of the structure to serve the purpose for which it is intended.

(*b*) Tests of materials and of concrete shall be made in accordance with the requirements of the American Society for Testing Materials as noted elsewhere in this chapter. The complete records of such tests shall be available for inspection during the progress of the work and for two years thereafter, and shall be preserved by the engineer or architect for that purpose.

202—Load tests

(*a*) When a load test is required, the member or portion of the structure under consideration shall be subject to a superimposed load equal to two times the live load plus one-half of the dead load. This load shall be left in position for a period of twenty-four hours before removal. If, during the test, or upon removal of the load, the member or portion of the structure shows evident failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or else, where lawful, a lower rating may be established. The structure shall be considered to have passed the test if the maximum deflection at the end of the twenty-four hour period does not exceed the value of D as given in the following formula:

$$D = \frac{L^2}{12{,}000\,t} \qquad (1)$$

in which all terms are expressed in the same units.

(*b*) If the deflection exceeds the value of D as given in formula (1), the construction shall be considered to have passed the test if within twenty-four hours after the removal of the load the residual deflection

does not exceed either forty percent of the maximum deflection observed under load or sixty percent of that given by formula (1). Under no circumstances will the construction be considered acceptable if the deflection under load exceeds three times that given by the formula.

203—Supervision

(*a*) Concrete work shall be supervised preferably by the engineer or architect responsible for its design, or by a competent representative responsible to him. A record shall be kept of such supervision, which record shall cover the quality and quantity of concrete materials, the mixing and placing of the concrete, the placing of the reinforcing steel and the general progress of the work. When the temperature falls below 40 degrees F, a complete record of the temperatures and of the protection given to the concrete while curing shall be kept. This record shall be available for inspection during the progress of the work and for two years thereafter and shall be preserved by the engineer or architect for that purpose.

204—Portland cement

(*a*) Portland cement shall conform to the "Standard Specifications for Portland Cement" (A.S.T.M. Designation: C150).

205—Concrete aggregates

(*a*) Concrete aggregates shall conform to the "Standard Specifications for Concrete Aggregates" (A.S.T.M. Designation: C33), or to the "Standard Specifications for Lightweight Aggregates for Concrete" (A.S.T.M. Designation: C130), except that aggregates failing to meet these specifications but which have been shown by special test or actual service to produce concrete of the required quality may be used under Section 302(a) Method 2, where authorized by the Commissioner of Buildings.

(*b*) The maximum size of the aggregate shall be not larger than one-fifth of the narrowest dimension between sides of the forms of the member for which the concrete is to be used nor larger than three-fourths of the minimum clear spacing between reinforcing bars.

206—Water

(*a*) Water used in mixing concrete shall be clean, and free from injurious amounts of oils, acids, alkalis, organic materials, or other deleterious substances.

207—Metal reinforcement

(*a*) Reinforcing bars shall conform to the requirements of the "Standard Specifications for Billet-Steel Bars for Concrete Reinforcement"

(ASTM Designation: A15), "Standard Specifications for Rail-Steel Bars for Concrete Reinforcement" (ASTM Designation: A16), or "Standard Specifications for Axle-Steel Bars for Concrete Reinforcement" (ASTM Designation: A160). Deformations on deformed bars shall conform to "Standard Specifications for Minimum Requirements for the Deformations of Deformed Bars for Concrete Reinforcement" (ASTM Designation: A305).

(*b*) Cold-drawn wire or welded wire fabric for concrete reinforcement shall conform to the requirements of the "Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (A. S. T. M. Designation: A82), or "Standard Specifications for Welded Steel Wire Fabric for Concrete Reinforcement" (A.S.T.M. Designation: A185).

(*c*) Structural steel shall conform to the requirements of the "Standard Specifications for Steel for Bridges and Buildings" (A.S.T.M. Designation: A7).

(*d*) Cast-iron sections for composite columns shall conform to the "Standard Specifications for Cast Iron Pit-Cast Pipe for Water and Other Liquids" (A.S.T.M. Designation: A44).

208—Storage of materials

(*a*) Cement and aggregates shall be stored in such a manner as to prevent deterioration or intrusion of foreign matter. Any material which has deteriorated or which has been damaged shall not be used for concrete.

CHAPTER 3—CONCRETE QUALITY AND ALLOWABLE STRESSES

300—Notation

f_c = Compressive unit stress in extreme fiber of concrete in flexure.
f'_c = Compressive strength of concrete at age of 28 days unless otherwise specified.
f_r = Compressive unit stress in the metal core of a composite column.
f_s = Tensile unit stress in longitudinal reinforcement; nominal allowable stress in vertical column reinforcement.
f_v = Tensile unit stress in web reinforcement.
n = Ratio of modulus of elasticity of steel to that of concrete.
u = Bond stress per unit of surface area of bar.
v = Shearing unit stress.
v_c = Shearing unit stress permitted on the concrete.

301—Concrete quality

(*a*) For the design of reinforced concrete structures, the value of f'_c used for determining the allowable stresses as stipulated in Section 305 shall be based on the specified minimum 28-day compressive strength of the concrete, or on the specified minimum compressive strength at the earlier age at which the concrete may be expected to receive its full load. All plans, submitted for approval or used on the job, shall clearly show the assumed strength of concrete at a specified age for which all parts of the structure were designed.

(*b*) No concrete exposed to the action of freezing weather shall have a water content exceeding six gallons per sack of cement.

302—Methods for determining strength of concrete

(*a*) The determination of the proportions of cement, aggregate and water to attain the required strengths shall be made by one of the following methods:

Method 1—Concrete made from average materials:

When no preliminary tests of the materials to be used are made, the water content per sack of cement shall not exceed the values in Table 302(*a*). Method 2 shall be employed when artificial aggregates or admixtures are used.

TABLE 302(a)—ASSUMED STRENGTH OF CONCRETE MIXTURES

Water content, U. S. gallons per 94-lb. sack of cement	Assumed compressive strength at 28 days, psi
7¼	2000
6¾	2500
6	3000
5	3750

NOTE—In interpreting this table, surface water carried by the aggregate must be included as part of the mixing water in computing the water content.

Method 2—Controlled concrete:

Water content other than shown in Table 302(*a*) may be used provided that the strength-quality of the concrete proposed for use in the structure shall be established by tests which shall be made in advance of the beginning of operations, using the consistencies suitable for the work and in accordance with the "Standard Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Laboratory" (A.S.T.M. Designation: C192) and with the "Standard Method of Test for Compressive Strength of Molded Concrete Cylinders (A.S.T.M. Designation: C39). A curve representing the relation between the water content and the average 28-day compressive strength or earlier strength at which the concrete is to receive its full working load, shall be established for a range of values including all the compressive strengths called for on the plans.

The curve shall be established by at least three points, each point representing average values from at least four test specimens. The maximum allowable water content for the concrete for the structure shall be as determined from this curve and shall correspond to a strength which is fifteen percent greater than that called for on the plans. No substitutions shall be made in the materials used on the work without additional tests in accordance herewith to show that the quality of the concrete is satisfactory.

303—Concrete proportions and consistency

(*a*) The proportions of aggregate to cement for any concrete shall be such as to produce a mixture which will work readily into the corners and angles of the forms and around reinforcement with the method of placing employed on the work, but without permitting the materials to segregate or excess free water to collect on the surface. The combined aggregates shall be of such composition of sizes that when separated on the No. 4 standard sieve, the weight passing the sieve (fine aggregate) shall not be less than thirty percent nor greater than fifty percent of the total, except that these proportions do not necessarily apply to lightweight aggregates.

(*b*) The methods of measuring concrete materials shall be such that the proportions can be accurately controlled and easily checked at any time during the work.* Measurement of materials for ready mixed concrete shall conform to the "Standard Specifications for Ready-Mixed Concrete" (A.S.T.M. Designation: C94).

304—Tests on concrete

(*a*) The Commissioner of Buildings may require a reasonable number of tests to be made during the progress of the work. Not less than three specimens shall be made for each test, nor less than one test for each 250 cu yd of concrete. Specimens shall be made and cured in accordance with the "Standard Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Field" (A.S.T.M. Designation: C31). Specimens shall be cured under laboratory conditions except when, in the opinion of the Commissioner of Buildings, there is a possibility of

*Wherever practicable such measurement shall be by weight rather than by volume.

the air temperature falling below 40 F, he may require the specimens to be cured under field conditions. Specimens shall be tested in accordance with the "Standard Method of Test for Compressive Strength of Molded Concrete Cylinders" (A.S.T.M. Designation: C39).

(*b*) The standard age of test shall be 28 days, but 7-day tests may be used provided that the relation between the 7- and 28-day strengths of the concrete is established by tests for the materials and proportions used.

(*c*) If the average strength of the laboratory control cylinders for any portion of the structure falls below the compressive strength required by the design, the Commissioner of Buildings shall have the right to order a change in the proportions or the water content of the concrete for the remaining portions of the structure. If the average strength of the cylinders cured on the job falls below the required strength, the Commissioner of Buildings shall have the right to require changes in the conditions of temperature and moisture necessary to secure the required strength.

(*d*) In addition, where there is question as to the quality of the concrete in the structure, the Commissioner of Buildings may require tests in accordance with the "Standard Methods of Securing, Preparing and Testing Specimens from Hardened Concrete for Compressive and Flexural Strengths", (A.S.T.M. Designation: C42) or order load tests as outlined in Section 202 for that portion of the structure where the questionable concrete has been placed.

305—Allowable unit stresses in concrete

(*a*) The unit stresses in pounds per square inch on concrete to be used in the design shall not exceed the values of Table 305(a) where f'_c equals the minimum specified compressive strength at 28 days, or at the earlier age at which the concrete may be expected to receive its full load.

306—Allowable unit stresses in reinforcement

Unless otherwise provided in this Code, steel for concrete reinforcement shall not be stressed in excess of the following limits:

(*a*) *Tension*

(f_s = Tensile unit stress in longitudinal reinforcement)
and (f_s = Tensile unit stress in web reinforcement)

20,000 psi for Rail-Steel Concrete Reinforcement Bars, Billet-Steel Concrete Reinforcement Bars of intermediate and hard grades, Axle-Steel Concrete Reinforcement Bars of intermediate and hard grades, and Cold-Drawn Steel Wire for Concrete Reinforcement.

18,000 psi for Billet-Steel Concrete Reinforcement Bars of structural grade, and Axle-Steel Concrete Reinforcement Bars of structural grade.

TABLE 305(a)—ALLOWABLE UNIT STRESSES IN CONCRETE

Description		Allowable unit stresses					
		For any strength of concrete in accordance with Section 302 $n = \frac{30{,}000}{f'_c}$	Maximum value, psi	For strength of concrete shown below			
				f'_c = 2000 psi n = 15	f'_c = 2500 psi n = 12	f'_c = 3000 psi n = 10	f'_c = 3750 psi n = 8
Flexure: f_c							
Extreme fiber stress in compression	f_c	$0.45f'_c$		900	1125	1350	1688
Extreme fiber stress in tension in plain concrete footings	f_t	$0.03f'_c$		60	75	90	113
Shear: v (as a measure of diagonal tension)							
Beams with no web reinforcement	v_c	$0.03f'_c$		60	75	90	113
Beams with properly designed web reinforcement	v	$0.12f'_c$		240	300	360	450
*Flat slabs at distance d from edge of column capital or drop panel	v_c	$0.03f'_c$		60	75	90	113
**Footings	v_c	$0.03f'_c$	75	60	75	75	75
Bond: u							
Deformed bars							
Top bars†	u	$0.07f'_c$	245	140	175	210	245
In 2-way footings (except top bars)	u	$0.08f'_c$	280	160	200	240	280
All others	u	$0.10f'_c$	350	200	250	300	350
Plain bars (must be hooked)							
Top bars	u	$0.03f'_c$	105	60	75	90	105
In 2-way footings (except top bars)	u	$0.036f'_c$	126	72	90	108	126
All others	u	$0.045f'_c$	158	90	113	135	158
Bearing: f_c							
On full area	f_c	$0.25f'_c$		500	625	750	938
On one-third area or less‡	f_c	$0.375f'_c$		750	938	1125	1405

*See Section 807. **See Sections 905(a) and 808(a).
†Top bars are horizontal bars so placed that more than 12 in. of concrete is cast in the member below the bar.
‡The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

(b) *Tension in one-way slabs of not more than 12-ft span*

(f_s = Tensile unit stress in main reinforcement).

For the main reinforcement, ⅜ inch or less in diameter, in one-way slabs, 50 percent of the minimum yield point specified in the Standard Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 psi.

(c) *Compression, vertical column reinforcement*

(f_s = Nominal allowable stress in vertical column reinforcement).

Forty percent of the minimum yield point specified in the Standard Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 psi.

(f_r = Allowable unit stress in the metal core of composite and combination columns):

Structural steel sections	16,000 psi
Cast iron sections	10,000 psi
Steel pipe	See limitations of Section 1106(b)

(*d*) *Compression, flexural members*

For compression reinforcement in flexural members see Section 706(b).

CHAPTER 4—MIXING AND PLACING CONCRETE

401—Preparation of equipment and place of deposit

(*a*) Before placing concrete, all equipment for mixing and transporting the concrete shall be cleaned, all debris and ice shall be removed from the spaces to be occupied by the concrete, forms shall be thoroughly wetted (except in freezing weather) or oiled, and masonry filler units that will be in contact with concrete shall be well drenched (except in freezing weather), and the reinforcement shall be thoroughly cleaned of ice or other coatings.

(*b*) Water shall be removed from place of deposit before concrete is placed unless otherwise permitted by the Commissioner of Buildings.

402—Mixing of concrete

(*a*) Unless otherwise authorized by the Commissioner of Buildings, the mixing of concrete shall be done in a batch mixer of approved type.

(*b*) The concrete shall be mixed until there is a uniform distribution of the materials and shall be discharged completely before the mixer is recharged.

(*c*) For job mixed concrete, the mixer shall be rotated at a speed recommended by the manufacturers and mixing shall be continued for at least one minute after all materials are in the mixer. A longer mixing period may be required for mixers larger than one cubic yard capacity.

(*d*) Ready-mixed concrete shall be mixed and delivered in accordance with the requirements set forth in the "Standard Specifications for Ready-Mixed Concrete" (A.S.T.M. Designation: C94).

403—Conveying

(*a*) Concrete shall be conveyed from the mixer to the place of final deposit by methods which will prevent the separation or loss of the materials.

(*b*) Equipment for chuting, pumping and pneumatically conveying concrete shall be of such size and design as to insure a practically continuous flow of concrete at the delivery end without separation of the materials.

404—Depositing

(*a*) Concrete shall be deposited as nearly as practicable in its final position to avoid segregation due to rehandling or flowing. The concreting shall be carried on at such a rate that the concrete is at all times plastic and flows readily into the spaces between the bars. No concrete that has partially hardened or been contaminated by foreign materials shall be deposited on the work, nor shall retempered concrete be used.

(*b*) When concreting is once started, it shall be carried on as a continuous operation until the placing of the panel or section is completed.

The top surface shall be generally level. When construction joints are necessary, they shall be made in accordance with Section 508.

(*c*) All concrete shall be thoroughly compacted by suitable means during the operation of placing, and shall be thoroughly worked around the reinforcement and embedded fixtures and into the corners of the forms. Vibrators may be used to aid in the placement of the concrete provided they are used under experienced supervision, and the forms are designed to withstand their action.

(*d*) Where conditions make compacting difficult, or where the reinforcement is congested, batches of mortar containing the same proportions of cement to sand as used in the concrete, shall first be deposited in the forms to a depth of at least one inch.

405—Curing

(*a*) In all concrete structures, concrete made with normal portland cement shall be maintained in a moist condition for at least the first seven days after placing and high-early-strength concrete shall be so maintained for at least the first three days.

406—Cold weather requirements

(*a*) Adequate equipment shall be provided for heating the concrete materials and protecting the concrete during freezing or near-freezing weather. No frozen materials or materials containing ice shall be used.

(*b*) All concrete materials and all reinforcement, forms, fillers and ground with which the concrete is to come in contact, shall be free from frost. Whenever the temperature of the surrounding air is below 40 degrees Fahrenheit, all concrete when placed in the forms shall have a temperature of between 60 and 90 degrees Fahrenheit and shall be maintained at a temperature of not less than 50 degrees Fahrenheit for at least 72 hours for normal concrete or 24 hours for high-early-strength concrete, or for as much more time as is necessary to insure proper rate of curing of the concrete. The housing, covering or other protection used in connection with curing shall remain in place and intact at least twenty-four hours after the artificial heating is discontinued. No dependence shall be placed on salt or other chemicals for the prevention of freezing. Manure, when used for protection, shall not be allowed to come into contact with the concrete.

CHAPTER 5—FORMS AND DETAILS OF CONSTRUCTION

501—Design of forms

(*a*) Forms shall conform to the shape, lines, and dimensions of the members as called for on the plans, and shall be substantial and sufficiently tight to prevent leakage of mortar. They shall be properly braced or tied together so as to maintain position and shape.

502—Removal of forms

(*a*) Forms shall be removed in such manner as to insure the complete safety of the structure. Where the structure as a whole is supported on shores, the removable floor forms, beam and girder sides, column and similar vertical forms may be removed after twenty-four hours, provided the concrete is sufficiently hard not to be injured thereby. In no case shall the supporting forms or shoring be removed until the members have acquired sufficient strength to support safely their weight and the load thereon. The results of suitable control tests may be used as evidence that the concrete has attained such sufficient strength.

503—Pipes, conduits, etc., embedded in concrete*

(*a*) Pipes which will contain liquid, gas or vapor at other than room temperature shall not be embedded in concrete necessary for structural stability or fire protection. Drain pipes and pipes whose contents will be under pressure greater than atmospheric pressure by more than one pound per square inch shall not be embedded in structural concrete except in passing through from one side to the other of a floor, wall or beam. Electric conduits and other pipes whose embedment is allowed shall not, with their fittings, displace that concrete of a column on which stress is calculated or which is required for fire protection, to greater extent than four per cent of the area of the cross section. Sleeves or other pipes passing through floors, walls or beams shall not be of such size or in such location as unduly to impair the strength of the construction; such sleeves or pipes may be considered as replacing structurally the displaced concrete, provided they are not exposed to rusting or other deterioration, are of uncoated iron or steel not thinner than standard wrought-iron pipe, have a nominal inside diameter not over two inches, and are spaced not less than three diameters on centers. Embedded pipes or conduits other than those merely passing through, shall not be larger in outside diameter than one-third the thickness of the slab, wall or beam in which they are embedded; shall not be spaced closer than three diameters on centers, nor so located as unduly to impair the strength of the construction. Circular uncoated or galvanized electric conduit of iron or steel may be considered as replacing the displaced concrete.

*Since this section was adopted a number of concrete floors have been built with pipes embedded for radiant heating which is prohibited by this section. The Committee is studying the problem for the purpose of revising the requirements to permit the safe use of such pipes in structural concrete.

504—Cleaning and bending reinforcement

(*a*) Metal reinforcement, at the time concrete is placed, shall be free from rust scale or other coatings that will destroy or reduce the bond. Bends for stirrups and ties shall be made around a pin having a diameter not less than two times the minimum thickness of the bar. Hooks shall conform to the requirements of Section 906. Bends for other bars shall be made around a pin having a diameter not less than six times the minimum thickness of the bar, except that for bars larger than one inch, the pin shall be not less than eight times the minimum thickness of the bar. All bars shall be bent cold.

505—Placing reinforcement

(*a*) Metal reinforcement shall be accurately placed and adequately secured in position by concrete or metal chairs or spacers. The minimum clear distance between parallel bars, except in columns, shall be equal to the nominal diameter of the bars. In no case shall the clear distance between bars be less than 1 in., nor less than one and one-third times the maximum size of the coarse aggregate. Where reinforcement in beams or girders is placed in two or more layers, the clear distance between layers shall not be less than 1 in., and the bars in the upper layers shall be placed directly above those in the bottom layer.

(*b*) When wire or other reinforcement, not exceeding one-fourth inch in diameter is used as reinforcement for slabs not exceeding ten ft. in span, the reinforcement may be curved from a point near the top of the slab over the support to a point near the bottom of the slab at mid-span; provided such reinforcement is either continuous over, or securely anchored to the support.

506—Splices in reinforcement

(*a*) In slabs, beams and girders, splices of reinforcement at points of maximum stress shall generally be avoided. Splices shall provide sufficient lap to transfer the stress between bars by bond and shear. The clear distance between bars shall also apply to the clear distance between a contact splice and adjacent contact splices or bars.

(*b*) Splices in the reinforcement of columns are specified in Section 1103(*c*).

507—Concrete protection for reinforcement

(*a*) The reinforcement of footings and other principal structural members in which the concrete is deposited against the ground shall have not less than three inches of concrete between it and the ground

contact surface. If concrete surfaces after removal of the forms are to be exposed to the weather or be in contact with the ground, the reinforcement shall be protected with not less than two inches of concrete for bars more than $\frac{5}{8}$ inch in diameter and one and one-half inches for bars $\frac{5}{8}$ inch or less in diameter.

(*b*) The concrete protective covering for reinforcement at surfaces not exposed directly to the ground or weather shall be not less than three-fourths inch for slabs and walls; and not less than one and one-half inches for beams, girders and columns. In concrete joist floors in which the clear distance between joists is not more than thirty inches, the protection of reinforcement shall be at least three-fourths inch.

(*c*) If the general code of which this Code forms a part specifies, as fire-protective covering of the reinforcement, thicknesses of concrete greater than those given in this section, then such greater thicknesses shall be used.

(*d*) Concrete protection for reinforcement shall in all cases be at least equal to the diameter of round bars, and one and one-half times the side dimension of square bars.

(*e*) Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion by concrete or other adequate covering.

508—Construction joints

(*a*) Joints not indicated on the plans shall be so made and located as to least impair the strength of the structure. Where a joint is to be made, the surface of the concrete shall be thoroughly cleaned and all laitance removed. In addition to the foregoing, vertical joints shall be thoroughly wetted, and slushed with a coat of neat cement grout immediately before placing of new concrete.

(*b*) At least two hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs supported thereon. Beams, girders, brackets, column capitals, and haunches shall be considered as part of the floor system and shall be placed monolithically therewith.

(*c*) Construction joints in floors shall be located near the middle of the spans of slabs, beams, or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. In this last case provision shall be made for shear by use of inclined reinforcement.

CHAPTER 6—DESIGN—GENERAL CONSIDERATIONS

600—Notation

f'_c = Compressive strength of concrete at age of 28 days, unless otherwise specified.

n = Ratio of modulus of elasticity of steel to that of concrete = $\frac{E_s}{E_c}$; assumed as equal to $\frac{30{,}000}{f'_c}$.

601—Assumptions

(*a*) The design of reinforced concrete members shall be made with reference to allowable stresses and safe loads. The accepted theory of flexure as applied to reinforced concrete shall be applied to all members resisting bending. The following assumptions shall be made:

1. The steel takes all the tensile stress.

2. In determining the ratio n for design purposes, the modulus of elasticity for the concrete shall be assumed as 1000 f'_c, and that for steel as 30,000,000 p.s.i.

602—Design loads

(*a*) The provisions for design herein specified are based on the assumption that all structures shall be designed for all dead- and live-loads coming upon them, the live-loads to be in accordance with the general requirements of the building code of which this forms a part, with such reductions for girders and lower story columns as are permitted therein.

603—Resistance to wind forces

(*a*) The resisting elements in structures required to resist wind forces shall be limited to the integral structural parts.

(*b*) The moments, shears, and direct stresses resulting from wind forces determined in accordance with recognized methods shall be added to the maximum stresses which obtain at any section for dead- and live-loads.

(*c*) In proportioning the component parts of the structure for the maximum combined stresses, including wind stresses, the unit stresses shall not exceed the stresses allowed for combined live- and dead-loads only by more than one-third. The structural members and their connections shall be so proportioned as to provide suitable rigidity of structure.

CHAPTER 7—FLEXURAL COMPUTATIONS

700—Notation

b = Width of rectangular flexural member or width of flanges for T and I sections.

b' = Width of web in T and I flexural members.

d = Depth from compression face of beam or slab to centroid of longitudinal tensile reinforcement; the diameter of a round bar or side of a square bar.

E = The modulus of elasticity of concrete in compression.

h = Unsupported length of a column.

I = Moment of inertia of a section about the neutral axis for bending.

K = The stiffness factor, that is, the moment of inertia divided by the span.

l = Span length of slab or beam.

l' = Clear span for positive moment and shear and the average of the two adjacent clear spans for negative moment (See Section 701).

t_1 = Minimum total thickness of slab.

w = Uniformly distributed load per unit of length of beam or per unit area of slab.

701—General requirements

(*a*) All members of frames or continuous construction shall be designed to resist at all sections the maximum moments and shears produced by dead load, live load and wind load, as determined by the theory of elastic frames in which the simplified assumptions of Section 702 may be used.

(*b*) Approximate methods of frame analysis are satisfactory for buildings of usual types of construction, spans and story heights.

(*c*) In the case of two or more approximately equal spans (the larger of two adjacent spans not exceeding the shorter by more than 20 per cent) with loads uniformly distributed, where the unit live load does not exceed three times the unit dead load, design for the following moments and shears is satisfactory:

Positive moment at center of span

End spans ..... $\frac{1}{14}\ wl'^2$

Interior Spans ..... $\frac{1}{16}\ wl'^2$

Negative moment at exterior face of first interior support

Two spans ..... $\frac{1}{9}\ wl'^2$

More than two spans ..... $\frac{1}{10}\ wl'^2$

Negative moment at other faces of interior supports ..... $\frac{1}{11}\ wl'^2$

Negative moment at face of all supports for, (a) slabs with spans not exceeding ten feet, and (b) beams and girders where ratio of sum of column stiffnesses to beam stiffness exceeds eight ..... $\frac{1}{12}\ wl'^2$

Shear in end members at first interior support ..... $1.15\ \frac{wl'}{2}$

Shear at other supports ..... $\frac{wl'}{2}$

702—Conditions of design*

(*a*) *Arrangement of live load*

1. The live load may be considered to be applied only to the floor under consideration, and the far ends of the columns may be assumed as fixed.

2. Consideration may be limited to combinations of dead load on all spans with full live load on two adjacent spans and with full live load on alternate spans.

(*b*) *Span length*

1. The span length, l, of members that are not built integrally with their supports shall be the clear span plus the depth of the slab or beam but shall not exceed the distance between centers of supports.

2. In analysis of continuous frames, center to center distances, l and h, may be used in the determination of moments. Moments at faces of supports may be used for design of beams and girders.

3. Solid or ribbed slabs with clear spans of not more than ten feet that are built integrally with their supports may be designed as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and the width of beams otherwise neglected.

(*c*) *Stiffness*

1. The relative stiffness, K, of a member with constant I is defined as EI divided by l or h.†

*Chapter 7 deals with floor members only. For moments in columns see Section 1108.

†Where displacements are involved the absolute value of K should be used. For members with a constant cross-section the absolute value of K equals $4EI/l$.

2. In computing the value of I of slabs, beams, girders, and columns, the reinforcement may be neglected. In T-shaped sections allowance shall be made for the effect of flange.

3. Any reasonable assumption may be adopted as to relative stiffness of columns and of floor system. The assumption made shall be consistent throughout the analysis.

(*d*) *Haunched Floor Members*

1. When members are widened near the supports, the additional width may be neglected in computing moments, but may be considered as resisting the resulting moments and shears.

2. When members are deepened near the supports, they may be analyzed as members of constant depth provided the minimum depth only is considered as resisting the resulting moments; otherwise an analysis taking into account the variation in depth is required. In any case, the actual depth may be considered as resisting shear.

(*e*) *Limitations*

1. Wherever at any section positive reinforcement is indicated by analysis, the amount provided shall be not less than .005 $b'd$ except in slabs of uniform thickness. (Use b instead of b' for rectangular flexural members.)

2. Not less than 0.005 $b'd$ of negative reinforcement shall be provided at the outer end of all members built integrally with their supports. (Use b instead of b' for rectangular flexural members.)

3. Where analysis indicates negative reinforcement along the full length of a span, the reinforcement need not be extended beyond the point where the required amount is 0.0025 $b'd$ or less. (Use b instead of b' for rectangular flexural members.)

4. In slabs of uniform thickness the minimum amount of reinforcement in the direction of the span shall be:

For structural, intermediate and hard grades and rail steel . . 0.0025 bd

For steel having a minimum yield point of 56,000 psi . . . . . 0.002 bd

703—Depth of beam or slab

(*a*) The depth of the beam or slab shall be taken as the distance from the centroid of the tensile reinforcement to the compression face of the structural members. Any floor finish not placed monolithically with the floor slab shall not be included as a part of the structural member. When the finish is placed monolithically with the structural slab in buildings of the warehouse or industrial class, there shall be placed an additional depth of one-half inch over that required by the design of the member.

704—Distance between lateral supports

(*a*) The clear distance between lateral supports of a beam shall not exceed thirty-two times the least width of compression flange.

705—Requirements for T-beams

(*a*) In T-beam construction the slab and beam shall be built integrally or otherwise effectively bonded together. The effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth of the span length of the beam, and its overhanging width on either side of the web shall not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

(*b*) For beams having a flange on one side only, the effective overhanging flange width shall not exceed one-twelfth of the span length of the beam, nor six times the thickness of the slab, nor one-half the clear distance to the next beam.

(*c*) Where the principal reinforcement in a slab which is considered as the flange of a T-beam (not a joist in concrete joist floors) is parallel to the beam, transverse reinforcement shall be provided in the top of the slab. This reinforcement shall be designed to carry the load on the portion of the slab required for the flange of the T-beam. The flange shall be assumed to act as a cantilever. The spacing of the bars shall not exceed five times the thickness of the flange, nor in any case eighteen inches.

(*d*) Provision shall be made for the compressive stress at the support in continuous T-beam construction, care being taken that the provisions of Section 505 relating to the spacing of bars, and 404(d), relating to the placing of concrete shall be fully met.

(*e*) The overhanging portion of the flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

(*f*) Isolated beams in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web and a total flange width not more than four times the web thickness.

706—Compression steel in flexural members

(*a*) Compression steel in beams or girders shall be anchored by ties or stirrups not less than ¼ inch in diameter spaced not farther apart than 16 bar diameters, or 48 tie diameters. Such stirrups or ties shall be used throughout the distance where the compression steel is required.

(*b*) The effectiveness of compression reinforcement in resisting bending may be taken at twice the value indicated from the calculations assuming a straight-line relation between stress and strain and the

modular ratio given in Section 601, but not of greater value than the allowable stress in tension.

707—Shrinkage and temperature reinforcement

(*a*) Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided in floor and roof slabs where the principal reinforcement extends in one direction only. Such reinforcement shall provide for the following minimum ratios of reinforcement area to concrete area bd, but in no case shall such reinforcing bars be placed farther apart than five times the slab thickness nor more than eighteen inches:

Floor slabs where plain bars are used	0.0025
Floor slabs where deformed bars are used	0.002
Floor slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than twelve inches	0.0018
Roof slabs where plain bars are used	0.003
Roof slabs where deformed bars are used	0.0025
Roof slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than twelve inches	0.0022

708—Concrete joist floor construction

(*a*) Concrete joist floor construction consists of concrete joists and slabs placed monolithically with or without burned clay or concrete tile fillers. The joists shall not be farther apart than thirty inches face to face. The ribs shall be straight, not less than four inches wide, and of a depth not more than three times the width.

(*b*) When burned clay or concrete tile fillers, of material having a unit compressive strength at least equal to that of the designed strength of the concrete in the joists are used, and the fillers are so placed that the joints in alternate rows are staggered, the vertical shells of the fillers in contact with the joists may be included in the calculations involving shear or negative bending moment. No other portion of the fillers may be included in the design calculations.

(*c*) The concrete slab over the fillers shall be not less than one and one-half inches in thickness, nor less in thickness than one-twelfth of the clear distance between joists. Shrinkage reinforcement shall be provided in the slab at right angles to the joists as required in Section 707, substituting t_1, (total thickness of slab) for d.

(*d*) Where removable forms or fillers not complying with (*b*) are used, the thickness of the concrete slab shall not be less than one-twelfth of the clear distance between joists and in no case less than two inches.

Such slab shall be reinforced at right angles to the joists with at least the amount of reinforcement required for flexure, giving due consideration to concentrations, if any, but in no case shall the reinforcement be less than that required by Section 707, considering t_1 as the full thickness of the slab.

(*e*) When the finish used as a wearing surface is placed monolithically with the structural slab in buildings of the warehouse or industrial class, the thickness of the concrete over the fillers shall be one-half inch greater than the thickness used for design purposes.

(*f*) Where the slab contains conduits or pipes, the thickness shall not be less than one inch plus the total over-all depth of such conduits or pipes at any point. Such conduits or pipes shall be so located as not to impair the strength of the construction.

(*g*) Shrinkage reinforcement shall not be required in the slab parallel to the joists.

709—Floors with supports on four sides*

(*a*) This construction, consisting of floors reinforced in two directions and supported on four sides, includes solid reinforced concrete slabs; concrete joists with fillers of hollow concrete units or clay tile, with or without concrete top slabs; and concrete joists with top slabs placed monolithically with the joists. The slab shall be supported by walls or beams on all sides and if not securely attached to supports, shall be reinforced as specified in 709(*b*).

(*b*) Where the slab is not securely attached to the supporting beams or walls, special reinforcement shall be provided at exterior corners in both the bottom and top of the slab. This reinforcement shall be provided for a distance in each direction from the corner equal to 1/5 the longest span. The reinforcement in the top of the slab shall be parallel to the diagonal from the corner. The reinforcement in the bottom of the slab shall be at right angles to the diagonal or may be of bars in two directions parallel to the sides of the slab. The reinforcement in each band shall be of equivalent size and spacing to that required for the maximum positive moment in the slab.

(*c*) The slab and its supports shall be designed by approved methods which shall take into account the effect of continuity at supports, the ratio of length to width of slab and the effect of two-way action.

(*d*) In no case shall the slab thickness be less than 4 in. nor less than the perimeter of the slab divided by 180. The spacing of reinforcement shall be not more than 3 times the slab thickness and the ratio of reinforcement shall be at least 0.0025.

*The requirements of this section are satisfied by either of the methods of design which follow this section

METHOD 1

Notation—

L = Length of clear span
L_1 = Length of clear span in the direction normal to L
g = Ratio of span between lines of inflection to L in the direction of span L, when span L only is loaded.
g_1 = Ratio of span between lines of inflection to L_1 in the direction of span L_1, when span L_1 only is loaded.
$r = \frac{gL}{g_1 L_1}$
w = Total uniform load per sq. ft.
W = Total uniform load between opposite supports on slab strip of any width or tota slab load on beam when considered as one-way construction.
x = Ratio of distance from support to any section of slab or beam, to span L or L_1.
B = Bending moment coefficient for one-way construction.
C = Factor modifying bending moments prescribed for one-way construction for use in proportioning the slabs and beams in the direction of L of slabs supported on four sides.
C_a = Ratio of the shear at any section of a slab strip distant xL from the support to the total load W on the strip in direction of L.
C_b = Ratio of the shear at any section of a beam distant xL from the support to the total load W on the beam in the direction of L.

W_1, C_1, C_{a1}, C_{b1}, are corresponding values of W, C, C_a, C_b, for slab strip or beam in direction of L_1.

(a) *Lines of inflection for determination of r*—The lines of inflection shall be determined by elastic analysis of the continuous structure in each direction, when only the span under consideration is loaded.

When the span L or L_1 is at least 2/3 and at most 3/2 of the adjacent continuous span or spans, the values of g or g_1 may be taken as 0.87 for exterior spans and 0.76 for interior spans. (See Fig. 1).

For spans discontinuous at both ends, g or g_1 shall be taken as unity.

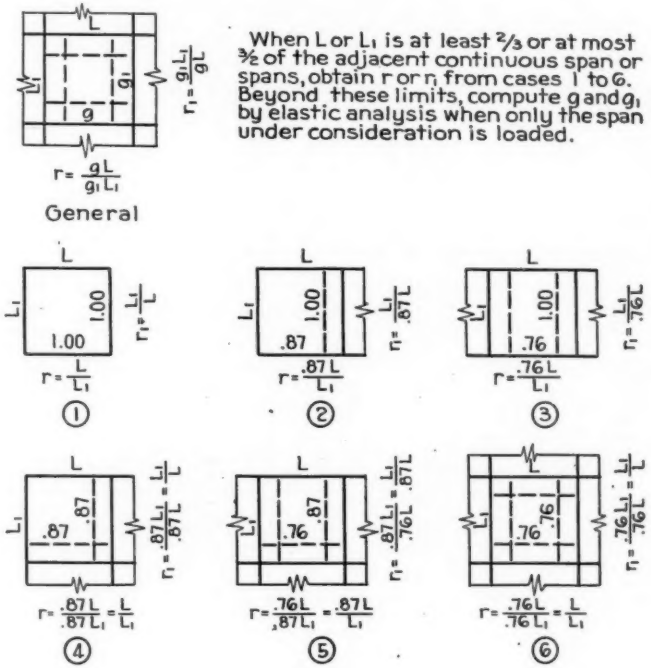
When L or L1 is at least 2/3 or at most 3/2 of the adjacent continuous span or spans, obtain r or r1 from cases 1 to 6. Beyond these limits, compute g and g1 by elastic analysis when only the span under consideration is loaded.
General
1.00
.87
.76
1
2
3
4
5
6

Fig. 1

(b) *Bending moments and shear*—Bending moments shall be determined in each direction with the coefficients prescribed for one-way construction in Sections 701 and 702 and modified by factor C or C_1 from Tables 1 or 2.

	In L Direction	In L_1 Direction
B.M. for slab strip	$M = CBWL$	$M_1 = C_1BW_1L_1$
B.M. for beam	$M = (1\text{-}C)BWL$	$M_1 = (1\text{-}C_1)\ BW_1L_1$

When the coefficients prescribed in 701(c) are used, the average value of Cw or C_1w for the two spans adjacent to a support shall be used in determining the negative bending moment at the face of the support.

TABLE 1—SLABS

Upper figure / Lower figure		C, C_m					C / C_1
	$\frac{1}{r}$	Values of x					
r		0.0	.1	.2	.3	.4	
0.00	∞	.50 .00	.40 .00	.30 .00	.20 .00	.10 .00	1.00 .00
.50	2.00	.44 .06	.36 .03	.27 .02	.18 .00	.09 .00	.89 .06
.55	1.82	.43 .07	.33 .04	.23 .02	.15 .01	.07 .00	.79 .08
.60	1.67	.41 .09	.30 .05	.20 .03	.12 .01	.05 .00	.70 .10
.65	1.54	.39 .11	.28 .06	.18 .03	.10 .01	.04 .00	.64 .13
.70	1.43	.37 .13	.26 .08	.16 .04	.09 .01	.03 .00	.58 .15
.80	1.25	.33 .17	.22 .10	.13 .06	.07 .02	.02 .00	48 .21
.90	1.11	.29 .21	.19 .13	.11 .07	.05 .03	.01 .01	.40 .27
1.00	1.00	.25 .25	.16 .16	.09 .09	.04 .04	.01 .01	.33 .33
1.10	.91	.21 .29	.13 .19	.07 .11	.03 .05	.01 .01	.28 .39
1.20	.83	.18 .32	.11 .21	.06 .13	.02 .06	.00 .02	.23 .45
1.30	.77	.16 .34	.10 .23	.05 .14	.02 .07	.00 .03	.19 .51
1.40	.71	.13 .37	.08 .25	.04 .16	.02 .09	.00 .03	.16 .57
1.50	.67	.11 .39	.07 .27	.04 .17	.01 .10	.00 .04	.14 .61
1.60	.63	.10 .40	.06 .29	.03 .19	.01 .11	.00 .05	.12 .66
1.80	.55	.07 .43	.04 .33	.02 .23	.01 .15	.00 .07	.08 .79
2.00	.50	.06 .44	.03 .36	.02 .27	.00 .18	.00 .09	.06 .89
∞	0.00	.00 .50	.00 .40	.00 .30	.00 .20	.00 .10	.00 1.00

The shear at any section distant xL or xL_1 from supports shall be determined by modifying the total load on the slab strip or beam by the factors C_s, C_{s1}, C_b or C_{b1} taken from Tables 1 or 2.

	In L Direction	In L_1 Direction
Shear for Slab Strip	$V = C_sW$	$V_1 = C_{s1}W_1$
Shear for Beam	$V = C_bW$	$V_1 = C_{b1}W_1$

For spans where the end moments are unbalanced, shear values at any section shall be adjusted in accordance with Sections 701 and 702.

(c) *Arrangement of reinforcement:*

1. In any panel, the area of reinforcement per unit width in the long direction shall be at least one-third that provided in the short direction.

2. The area of positive moment reinforcement adjacent to a continuous edge only and for a width not exceeding one-fourth of the shorter dimension of the panel may be reduced 25 per cent.

3. At a non-continuous edge the area of negative moment reinforcement per unit width shall be at least one-half of that required for maximum positive moment.

TABLE 2—BEAMS

Upper figure = C_b, Lower figure = C_M							$1-C$ / $1-C_1$
	$\frac{1}{r}$	Values of x					
r		0.0	.1	.2	.3	.4	
0.00	∞	.00 / .50	.00 / .40	.00 / .30	.00 / .20	.00 / .10	.00 / 1.00
.50	2.00	.06 / .44	.04 / .37	.03 / .28	.02 / .20	.01 / .10	.11 / .94
.55	1.82	.07 / .43	.07 / .36	.07 / .28	.05 / .19	.03 / .10	.21 / .92
.60	1.67	.09 / .41	.10 / .35	.10 / .27	.08 / .19	.05 / .10	.30 / .90
.65	1.54	.11 / .39	.12 / .34	.12 / .27	.10 / .19	.06 / .10	.36 / .87
.70	1.43	.13 / .37	.14 / .32	.14 / .26	.11 / .19	.07 / .10	.42 / .85
.80	1.25	.17 / .33	.18 / .30	.17 / .24	.13 / .18	.08 / .10	.52 / .79
.90	1.11	.21 / .29	.21 / .27	.19 / .23	.15 / .17	.09 / .09	.60 / .73
1.00	1.00	.25 / .25	.24 / .24	.21 / .21	.16 / .16	.09 / .09	.67 / .67
1.10	.91	.29 / .21	.27 / .21	.23 / .19	.17 / .15	.09 / .09	.72 / .61
1.20	.83	.32 / .18	.29 / .19	.24 / .17	.18 / .14	.10 / .08	.77 / .55
1.30	.77	.34 / .16	.30 / .17	.25 / .16	.18 / .13	.10 / .07	.81 / .49
1.40	.71	.37 / .13	.32 / .15	.26 / .14	.18 / .11	.10 / .07	.84 / .43
1.50	.67	.39 / .11	.33 / .13	.26 / .13	.19 / .10	.10 / .06	.86 / .39
1.60	.63	.40 / .10	.34 / .11	.27 / .11	.19 / .09	.10 / .05	.88 / .34
1.80	.55	.43 / .07	.36 / .07	.28 / .07	.19 / .05	.10 / .03	.92 / .21
2.00	.50	.44 / .06	.37 / .04	.28 / .03	.20 / .02	.10 / .01	.94 / .11
∞	0.00	.50 / .00	.40 / .00	.30 / .00	.20 / .00	.10 / .00	1.00 / .00

METHOD 2

Notation—

C = Moment coefficient for two-way slabs as given in Table 3
m = Ratio of short span to long span for two-way slabs
S = Length of short span for two-way slabs
The span shall be considered as the center-to-center distance between supports or the clear span plus twice the thickness of slab, whichever value is the smaller.
w = Total uniform load per sq. ft.

(a) *Limitations*—A two-way slab shall be considered as consisting of strips in each direction as follows:

A middle strip one-half panel in width, symmetrical about panel center line and extending through the panel in the direction in which moments are considered.

A column strip one-half panel in width, occupying the two quarter-panel areas outside the middle strip.

Where the ratio of short to long span is less than 0.5, the middle strip in the short direction shall be considered as having a width equal to the difference between the long and short span, the remaining area representing the two column strips.

The critical sections for moment calculations are referred to as principal design sections and are located as follows:

For negative moment, along the edges of the panel at the faces of the supporting beams.

For positive moment, along the center lines of the panels.

(b) *Bending Moments*—The bending moments for the middle strips shall be computed from the formula

$$M = CwS^2$$

The average moments per foot of width in the column strip shall be two-thirds of the corresponding moments in the middle strip. In determining the spacing of the reinforcement in the column strip, the moment may be assumed to vary from a maximum at the edge of the middle strip to a minimum at the edge of the panel.

Where the negative moment on one side of a support is less than 80 per cent of that on the other side, two-thirds of the difference shall be distributed in proportion to the relative stiffnesses of the slabs.

TABLE 3—MOMENT COEFFICIENTS

Moments	Short span, Values of m						Long span, all values of m
	1.0	0.9	0.8	0.7	0.6	0.5 and less	
Case 1—Interior panels							
Negative moment at—							
Continuous edge	.033	.040	.048	.055	.063	.083	.033
Discontinuous edge	—	—	—	—	—	—	—
Positive moment at midspan	.025	.030	.036	.041	.047	.062	.025
Case 2—One edge discontinuous							
Negative moment at—							
Continuous edge	.041	.048	.055	.062	.069	.085	.041
Discontinuous edge	.021	.024	.027	.031	.035	.042	.021
Positive moment at midspan	.031	.036	.041	.047	.052	.064	.031
Case 3—Two edges discontinuous							
Negative moment at—							
Continuous edge	.049	.057	.064	.071	.078	.090	.049
Discontinuous edge	.025	.028	.032	.036	.039	.045	.025
Positive moment at midspan	.037	.043	.048	.054	.059	.068	.037
Case 4—Three edges discontinuous							
Negative moment at—							
Continuous edge	.058	.066	.074	.082	.090	.098	.058
Discontinuous edge	.029	.033	.037	.041	.045	.049	.029
Positive moment at midspan	.044	.050	.056	.062	.068	.074	.044
Case 5—Four edges discontinuous							
Negative moment at—							
Continuous edge	—	—	—	—	—	—	—
Discontinuous edge	.033	.038	.043	.047	.053	.055	.033
Positive moment at midspan	.050	.057	.064	.072	.080	.083	.050

(c) *Shear*—The shearing stresses in the slab may be computed on the assumption that the load is distributed to the supports in accordance with (*d*).

(d) *Supporting Beams*—The loads on the supporting beams for a two-way rectangular panel may be assumed as the load within the tributary areas of the panel bounded by the intersection of 45-degree lines from the corners with the median line of the panel parallel to the long side.

The bending moments may be determined approximately by using an equivalent uniform load per lineal foot of beam for each panel supported as follows:

For the short span, $\frac{wS}{3}$

For the long span, $\frac{wS}{3} \quad \frac{(3-m^2)}{2}$

710—Maximum spacing of principal slab reinforcement

(*a*) In slabs other than concrete joist floor construction or flat slabs, the principal reinforcement shall not be spaced farther apart than three times the slab thickness, nor shall the ratio of reinforcement be less than specified in Section 707(a).

CHAPTER 8—SHEAR AND DIAGONAL TENSION

800—Notation

A_v = Total area of web reinforcement in tension within a distance of s (measured in a direction parallel to that of the main reinforcement), or the total area of all bars bent up in any one plane.

α = Angle between inclined web bars and axis of beam.

b = Width of rectangular flexural member or width of flange for T and I sections.

b' = Width of web in T and I flexural members.

d = Depth from compression face of beam or slab to centroid of longitudinal tensile reinforcement.

f'_c = Compressive strength of concrete at age of 28 days unless otherwise specified.

f_v = Tensile unit stress in web reinforcement.

j = Ratio of distance between centroid of compression and centroid of tension to the depth d.

s = Spacing of stirrups or of bent bars in a direction parallel to that of the main reinforcement.

t_2 = Thickness of flat slab without drop panels, or the thickness of flat slab through the drop panels where such are used.

t_3 = Thickness of flat slab (with drop panels) at points outside the drop panel.

v = Shearing unit stress.

V = Total shear.

V' = Excess of the total shear over that permitted on the concrete.

801—Shearing unit stress

(*a*) The shearing unit stress v, as a measure of diagonal tension, in reinforced concrete flexural members shall be computed by formula (12):

$$v = \frac{V}{bjd} \qquad (12)$$

(*b*) For beams of I or T section, b' shall be substituted for b in formula (12).

(*c*) In concrete joist floor construction, where burned clay or concrete tile are used, b' may be taken as a width equal to the thickness of the concrete web plus the thicknesses of the vertical shells of the concrete or burned clay tile in contact with the joist as in Section 708(b).

(*d*) When the value of the shearing unit stress computed by formula (12) exceeds the shearing unit stress v_c permitted on the concrete of an unreinforced web (see Section 305), web reinforcement shall be provided to carry the excess.

802—Types of web reinforcement

(*a*) Web reinforcement may consist of:

1. Stirrups or web reinforcement bars perpendicular to the longitudinal steel.

2. Stirrups or web reinforcement bars welded or otherwise rigidly attached to the longitudinal steel and making an angle of 30 degrees or more thereto.

3. Longitudinal bars bent so that the axis of the inclined portion of the bar makes an angle of 15 degrees or more with the axis of the longitudinal portion of the bar.

4. Special arrangements of bars with adequate provisions to prevent slip of bars or splitting of the concrete by the reinforcement (see Section 804(f)).

(*b*) Stirrups or other bars to be considered effective as web reinforcement shall be anchored at both ends, according to the provisions of Section 904.

803—Stirrups

(*a*) The area of steel required in stirrups placed perpendicular to the longitudinal reinforcement shall be computed by formula (13).

$$A_v = \frac{V's}{f_v jd} \quad \text{.....(13)}$$

(*b*) Inclined stirrups shall be proportioned by formula (15) (Section 804(d).)

(*c*) Stirrups placed perpendicular to the longitudinal reinforcement shall not be used alone as web reinforcement when the shearing unit stress (v) exceeds $0.08f'_c$.

804—Bent bars

(*a*) When the web reinforcement consists of a single bent bar or of a single group of bent bars the required area of such bars shall be computed by formula (14).

$$A_v = \frac{V'}{f_v \sin \alpha} \quad \text{.....(14)}$$

(*b*) In formula (14) V' shall not exceed $0.040\, f'_c\, bjd$.

(*c*) Only the center three-fourths of the inclined portion of such bar, or group of bars, shall be considered effective as web reinforcement.

(*d*) Where there is a series of parallel bent bars, the required area shall be determined by formula (15).

$$A_v = \frac{V's}{f_v jd\,(\sin \alpha + \cos \alpha)} \quad \text{.....(15)}$$

(*e*) When bent bars, having a radius of bend of not more than three times the diameter of the bar are used alone as web reinforcement, the allowable shearing unit stress shall not exceed $0.060\,f'_c$. This shearing unit stress may be increased at the rate of $0.01\,f'_c$ for each increase of four bar diameters in the radius of bend until the maximum allowable shearing unit stress is reached. (See Section 305(a).)

(*f*) The shearing unit stress permitted when special arrangements of bars are employed shall be that determined by making comparative tests, to destruction, of specimens of the proposed system and of similar specimens reinforced in conformity with the provisions of this code, the same factor of safety being applied in both cases.

805—Combined web reinforcement

(*a*) Where more than one type of reinforcement is used to reinforce the same portion of the web, the total shearing resistance of this portion of the web shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete shall be included only once, and no one type of reinforcement shall be assumed to resist more than $\frac{2V'}{3}$.

806—Spacing of web reinforcement

(*a*) Where web reinforcement is required it shall be so spaced that every 45 degree line (representing a potential crack) extending from the mid-depth of the beam to the longitudinal tension bars shall be crossed by at least one line of web reinforcement. If a shearing unit stress in excess of $0.06\,f'_c$ is used, every such line shall be crossed by at least two such lines of web reinforcement.

807—Shearing stress in flat slabs

(*a*) In flat slabs, the shearing unit stress on a vertical section which lies at a distance $t_2 - 1\frac{1}{2}$ in. beyond the edge of the column capital and parallel or concentric with it, shall not exceed the following values when computed by formula (12) (in which d shall be taken as $t_2 - 1\frac{1}{2}$ in.):

1. $0.03\,f'_c$, when at least 50 percent of the total negative reinforcement in the column strip passes directly over the column capital.

2. $0.025\,f'_c$, when 25 percent or less of the total negative reinforcement in the column strip passes directly over the column capital.

3. For intermediate percentages, intermediate values of the shearing unit stress shall be used.

(*b*) In flat slabs, the shearing unit stress on vertical sections which lie at a distance $t_2 - 1\frac{1}{2}$ in. beyond the edge of the drop panel and

parallel with them shall not exceed $0.03 f'_c$ when computed by formula (12) (in which d shall be taken as $t_3 - 1\frac{1}{2}$ in.). At least 50 percent of the cross-sectional area of the negative reinforcement in the column strip must be within the width of strip directly above the drop panel.

808—Shear and diagonal tension in footings

(*a*) In isolated footings the shearing unit stress computed by formula (12) on the critical section (see 1205(a)), shall not exceed $0.03 f'_c$, nor in any case shall it exceed 75 psi.

CHAPTER 9—BOND AND ANCHORAGE

900—Notation

d = Depth from compression face of beam or slab to centroid of longitudinal tensile reinforcement.
f'_c = Compressive strength of concrete at age of 28 days unless otherwise specified.
j = Ratio of distance between centroid of compression and centroid of tension to the depth d.
Σo = Sum of perimeters of bars in one set.
u = Bond stress per unit of surface area of bar.
V = Total shear.

901—Computation of bond stress in beams

(*a*) In flexural members in which the tensile reinforcement is parallel to the compression face, the bond stress at any cross section shall be computed by formula (16).

$$u = \frac{V}{\Sigma o\, jd} \qquad (16)$$

in which V is the shear at that section.

(*b*) Adequate end anchorage shall be provided for the tensile reinforcement in all flexural members to which formula (16) does not apply, such as sloped, stepped or tapered footings, brackets or beams in which the tensile reinforcement is not parallel to the compression face.

902—Anchorage requirements

(*a*) Tensile negative reinforcement in any span of a continuous, restrained or cantilever beam, or in any member of a rigid frame shall be adequately anchored by bond, hooks or mechanical anchors in or through the supporting member. Within any such span every reinforcing bar, whether required for positive or negative reinforcement, shall be extended at least 12 diameters beyond the point at which it is no longer needed to resist stress. The maximum tension in any bar must be developed by bond on a sufficient straight or bent embedment or by other anchorage (see Section 906). If preferred, the bar may be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the bar and made continuous with the reinforcement which resists moment of opposite sign.

(*b*) Of the positive reinforcement in continuous beams not less than one-fourth the area shall extend along the same face of the beam into the support a distance of 6 in.

(*c*) In simple beams, or at the freely supported end of continuous beams, at least one-third the required positive reinforcement shall

extend along the same face of the beam into the support a distance of 6 in.

903—Plain bars in tension

Plain bars in tension shall terminate in standard hooks except that hooks shall not be required on the positive reinforcement at interior supports of continuous members.

904—Anchorage of web reinforcement

(*a*) Single separate bars used as web reinforcement shall be anchored at each end by one of the following methods:

1. Welding to longitudinal reinforcement.

2. Hooking tightly around the longitudinal reinforcement through 180 degrees.

3. Embedment above or below the mid-depth of the beam on the compression side, a distance sufficient to develop the stress to which the bar will be subjected at a bond stress of not to exceed $0.045 f'_c$ on plain bars nor $0.10 f'_c$ on deformed bars.

4. Standard hook (see Section 906(a)), considered as developing 10,000 psi, plus embedment sufficient to develop by bond the remainder of the stress to which the bar is subjected. The unit bond stress shall not exceed that specified in Table 305(a). The effective embedded length shall not be assumed to exceed the distance between the mid-depth of the beam and the tangent of the hook.

(*b*) The extreme ends of bars forming simple U or multiple stirrups shall be anchored by one of the methods of Section 904(a) or shall be bent through an angle of at least 90 degrees tightly around a longitudinal reinforcing bar not less in diameter than the stirrup bar, and shall project beyond the bend at least twelve diameters of the stirrup bar.

(*c*) The loops or closed ends of such stirrups shall be anchored by bending around the longitudinal reinforcement through an angle of at least 90 degrees, or by being welded or otherwise rigidly attached thereto.

(*d*) Hooking or bending stirrups or separate web reinforcement bars around the longitudinal reinforcement shall be considered effective only when these bars are perpendicular to the longitudinal reinforcement.

(*e*) Longitudinal bars bent to act as web reinforcement shall, in a region of tension, be continuous with the longitudinal reinforcement. The tensile stress in each bar shall be fully developed in both the upper and the lower half of the beam by one of the following methods:

1. As specified in Section 904(a) (3).
2. As specified in Section 904(a) (4).

3. By bond at a unit bond stress not exceeding $0.045 f'_c$ on plain bars nor $0.10 f'_c$ on deformed bars, plus a bend of radius not less than two times the diameter of the bar, plus an extension of the bar, parallel to the upper or lower surface of the beam, of not less than 12 diameters of the bar, terminating in a standard hook. This short radius bend, extension and hook shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 psi.

4. By bond, at a unit bond stress not exceeding $0.045 f'_c$ on plain bars nor $0.10 f'_c$ on deformed bars, plus a bend of radius not less than two times the diameter of the bar, parallel to the upper or lower surface of the beam and continuous with the longitudinal reinforcement. The short radius bend and continuity shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 psi.

5. The tensile unit stress at the beginning of a bend may be increased from 10,000 psi when the radius of bend is two bar diameters, at the rate of 1,000 psi tension for each increase of 1½ bar diameters in the radius of bend, provided that the length of the bar in the bend and extension is sufficient to develop this increased tensile stress by bond at the unit stresses given in Section 904(e) (3).

(*f*) In all cases web reinforcement shall be carried as close to the compression surface of the beam as fireproofing regulations and the proximity of other steel will permit.

905—Anchorage of bars in footing slabs

(*a*) Plain bars in footing slabs shall be anchored by means of standard hooks. The outer faces of these hooks and the ends of deformed bars shall be not less than 3 in. nor more than 6 in. from the face of the footing.

906—Hooks

(*a*) The terms "hook" or "standard hook" as used herein shall mean either

1. A complete semicircular turn with a radius of bend on the axis of the bar of not less than three and not more than six bar diameters, plus an extension of at least four bar diameters at the free end of the bar, or

2. A 90° bend having a radius of not less than four bar diameters plus an extension of twelve bar diameters.

Hooks having a radius of bend of more than six bar diameters shall be considered merely as extensions to the bars, and shall be treated as in section 904(e) (5).

(*b*) In general, hooks shall not be permitted in the tension portion of any beam except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

(*c*) No hook shall be assumed to carry a load which would produce a tensile stress in the bar greater than 10,000 psi.

(*d*) Hooks shall not be considered effective in adding to the compressive resistance of bars.

(*e*) Any mechanical device capable of developing the strength of the bar without damage to the concrete may be used in lieu of a hook. Tests must be presented to show the adequacy of such devices.

CHAPTER 10—FLAT SLABS—WITH SQUARE OR RECTANGULAR PANELS

1000—Notation

A = The distance from the center line of the column, in the direction of any span, to the intersection of a 45-degree diagonal line from the center of the column to the bottom of the flat slab or drop panel, where such line lies wholly within the column, capital, or bracket, provided such capital or bracket is structurally capable of resisting shears and moments without excessive unit stress. In no case shall A be greater than one-eighth the span in the direction considered.

A_{av} = Average of the two values of A for the two columns at the ends of a column strip, in the direction of the spans considered.

c = Diameter or width of column capital at the under side of the slab or drop panel. No portion of the column capital shall be considered for structural purposes which lies outside the largest right circular cone, with 90 degrees vertex angle, that can be included within the outlines of the column capital.

L = Span length of slab center to center of columns in the direction of which bending is considered.

M_o = Sum of the positive and the average negative bending moments at the critical design sections of a flat slab panel. See Section 1003(b).

W = Total dead and live load uniformly distributed over a single panel area.

W_{av} = The average of the total load on two adjacent panels.

x = Coefficient of span L which gives the distance from the center of column to the critical section for negative bending in design according to Section 1002(a).

1001—Scope

(*a*) The term flat slab shall mean a reinforced concrete slab supported by columns with or without flaring heads or column capitals, with or without depressed or drop panels and generally without beams or girders.

(*b*) Recesses or pockets in flat slab ceilings, located between reinforcing bars and forming cellular or two-way ribbed ceilings, whether left open or filled with permanent fillers, shall not prevent a slab from being considered a flat slab; but allowable unit stresses shall not be exceeded.

(c) This chapter provides for two methods of design of flat slab structures.

1. Any type of flat slab construction may be designed by application of the principles of continuity, using the method outlined in Section 1002, or using other recognized methods of elastic analysis. In either case, the design must be subject to the provisions of Sections 1005(a) and (c), 1006, 1008 and 1009.

2. The common cases of flat slab construction described in Section 1003 may be designed by the use of moment coefficients, given in Sections 1003 and 1004, and subject to the provisions of Sections 1005, 1006, 1007, 1008 and 1009.

1002—Design of flat slabs as continuous frames

(a) Except in the cases of flat slab construction where specified coefficients for bending may be used, as provided in Section 1003, bending and shear in flat slabs and their supports shall be determined by an analysis of the structure as a continuous frame, and all sections shall be proportioned to resist the moments and shears thus obtained. In the analysis, the following assumptions may be made:

1. The structure may be considered divided into a number of bents, each consisting of a row of columns and strips of supported slabs, each strip bounded laterally by the center line of the panel on either side of the row of columns. The bents shall be taken longitudinally and transversely of the building.

2. Each such bent may be analyzed in its entirety; or each floor thereof and the roof may be analyzed separately with its adjacent columns above and below, the columns being assumed fixed at their remote ends. Where slabs are thus analyzed separately, in bents more than four panels long, it may be assumed in determining the bending at a given support that the slab is fixed at any support two panels distant therefrom beyond which the slab continues.

3. The joints between columns and slabs may be considered rigid and this rigidity may be assumed to extend in the slabs a distance A from the center of the columns, and in the column to the intersection of the sides of the column and the 45 degree line defining A. The change in length of columns and slabs due to direct stress, and deflections due to shear, may be neglected. Where metal column capitals are used, account may be taken of their contributions to stiffness and resistance to bending and shear.

4. The supporting columns may be assumed free from settlement or lateral movement unless the amount thereof can be reasonably determined.

5. The moment of inertia of slab or column at any cross-section may be assumed to be that of the gross section of the concrete. Variation in the moments of inertia of the slabs and columns along their axes shall be taken into account.

6. Where the load to be supported is definitely known, the structure shall be analyzed for that load. Where the live load is variable but does not exceed three-quarters of the dead load, or the nature of the live load is such that all panels will be loaded simultaneously, the maximum bending may be assumed to obtain at all sections under full live load. Elsewhere, maximum positive bending near mid-span of a panel may be assumed to obtain under full live load in the panel and in alternate panels; and maximum negative bending at a support may be assumed to obtain under full live load in the adjacent panels only.

7. Where neither beams nor girders help to transfer the slab load to the supporting column, the critical section for negative bending may be assumed as not more than the distance xL from the column center, where

$$x = 0.073 + 0.57 \frac{A}{L} \quad \text{.....(17)}$$

In slabs supported by beams, girders, or walls, the critical section for negative bending shall be assumed at the face of such support.

8. The numerical sum of the maximum positive and the average maximum negative bending moments for which provision is made in the design in the direction of either side of a rectangular panel shall be assumed as not less than

$$\frac{1}{10} W_{av} L \left(1 - \frac{4A_{av}}{3L}\right)^2 \quad \text{.....(18)}$$

9. The bending at critical sections across the slabs of each bent may be apportioned between the column strip and middle strip, as defined in Section 1005, in the ratio of the specified coefficients which affect such apportionment in the special cases of flat slabs provided for in Section 1003.

10. The maximum bending in columns may be assumed to obtain under full live load in alternate panels. Columns shall be proportioned to resist the maximum bending combined with the maximum direct load consistent therewith; and for maximum direct load combined with the bending under full load, the direct load subject to allowable reductions. In computing moments in columns at any floor, the far ends of the columns may be considered fixed.

(*b*) The foregoing provisions outline the method to be followed in analyzing and designing flat slabs in the general case. In all instances

the design must conform to the requirements for panel strips and critical design sections, slab thickness and drop panels, capitals and brackets, arrangement of reinforcement and openings in flat slabs, as provided in Sections 1005(a) and (c), 1006, 1008 and 1009.

1003—Design of flat slabs by moment coefficients

(*a*) In those cases of flat slab construction which fall within the following limitations as to continuity and dimensions, the bending moments at critical sections may be determined by the use of specified coefficients as provided in Section 1004.

1. The ratio of length to width of panel does not exceed 1.33.
2. The slab is continuous for at least three panels in each direction.
3. The successive span lengths in each direction differ by not more than twenty percent of the shorter span.

(*b*) In such slabs, the numerical sum of the positive and negative bending moments in the direction of either side of an interior rectangular panel shall be assumed as not less than

$$M_o = 0.09\,WL\left(1 - \frac{2c}{3L}\right)^2 \qquad (19)$$

(*c*) Three-fourths of the width of the strip shall be taken as the width of the section in computing compression due to bending, except that, on a section through a drop panel, three-fourths of the width of the drop panel shall be taken. Account shall be taken of any recesses which reduce the compressive area. Tension reinforcement distributed over the entire strip shall be included in the computations.

(*d*) The design of slabs under the procedure given in this section is subject to the provisions of all subsequent sections of this chapter (Sections 1004 to 1009).

1004—Bending moment coefficients

(*a*) The bending moments at the critical sections of the middle and column strips of an interior panel shall be assumed as given in Table 1004(a).

(*b*) The bending moments at critical sections of strips, in an exterior panel, at right angles to the discontinuous edge, where the exterior supports consist of reinforced concrete columns or reinforced concrete bearing walls integral with the slab, the ratio of stiffness of the support to that of the slab being at least as great as the ratio of the live load to the dead load and not less than one, shall be assumed as given in Table 1004(b). Where a flat slab is so supported by a wall providing restraint at the discontinuous edge, the coefficient for negative bending at the edge shall be assumed more nearly equal in the column and middle strips,

TABLE 1004(a)—BENDING MOMENTS IN INTERIOR FLAT SLAB PANEL

With drop panel		
Column strip	Negative moment	0.50M$_o$
	Positive moment	0.20M$_o$
Middle strip	Negative moment	0.15M$_o$
	Positive moment	0.15M$_o$
Without drop panel		
Column strip	Negative moment	0.46M$_o$
	Positive moment	0.22M$_o$
Middle strip	Negative moment	0.16M$_o$
	Positive moment	0.16M$_o$

TABLE 1004(b)—BENDING MOMENTS IN EXTERIOR FLAT SLAB PANEL

With drop panel		
Column strip	Exterior negative	0.45M$_o$
	Positive moment	0.25M$_o$
	Interior negative	0.55M$_o$
Middle strip	Exterior negative	0.10M$_o$
	Positive moment	0.19M$_o$
	Interior negative	0.165M$_o$
Without drop panel		
Column strip	Exterior negative	0.41M$_o$
	Positive moment	0.28M$_o$
	Interior negative	0.50M$_o$
Middle strip	Exterior negative	0.10M$_o$
	Positive moment	0.20M$_o$
	Interior negative	0.176M$_o$

TABLE 1004(c)—BENDING MOMENTS IN PANELS WITH MARGINAL BEAMS OR WALLS

		Marginal beams with depth greater than 1½ times the slab thickness; or bearing wall.		Marginal beams with depth 1½ times the slab thickness or less.	
(a) Load to be carried by marginal beam or wall		Loads directly superimposed upon it plus a uniform load equal to one-quarter of the total live and dead panel load.		Loads directly superimposed upon it exclusive of any panel load.	
		With drop	Without drop	With drop	Without drop
(b) Moment to be used in the design of half column strip adjacent and parallel to marginal beam or wall.	Neg.	0.125M$_o$	0.115M$_o$	0.25M$_o$	0.23M$_o$
	Pos.	0.05M$_o$	0.055M$_o$	0.10M$_o$	0.11M$_o$
(c) Negative moment to be used in design of middle strip continuous across a beam or wall	Neg.	0.195M$_o$	0.208M$_o$	0.15M$_o$	0.16M$_o$

the sum remaining as given in Table 1004(b), but that for the column strip shall not be less than 0.30 M_o. Bending in middle strips parallel to a discontinuous edge, except in a corner panel, shall be assumed the same as in an interior panel. M_o shall be determined as provided in Section 1003(b) for an interior panel.

(*c*) The bending moments at critical sections of strips, in an exterior panel, at right angles to the discontinuous edge, where the exterior supports are masonry bearing walls or other construction which provide only negligible restraint to the slab, shall be assumed as given in Table 1004(b) with the following modifications.

1. On critical sections at the face of the exterior support, negative bending in each strip shall be assumed as 0.05 M_o.

2. The coefficients for positive bending shall be increased by forty percent.

3. The coefficients for negative bending at the first interior columns shall be increased thirty percent.

(*d*) The bending moments in panels with marginal beams or walls, in the strips parallel and close thereto, and in the beams, shall be determined upon the basis of assumptions presented in Table 1004(c).

(*e*) For design purposes any of the moment coefficients of Tables 1004(a), 1004(b), and 1004(c) may be varied by not more than six percent, but the numerical sum of the positive and negative moments in a panel shall not be taken as less than the amount specified.

(*f*) Panels supported by marginal beams on opposite edges shall be designed as solid one or two-way slabs to carry the entire panel load.

(*g*) The ratio of reinforcement in any strip shall not be less than 0.0025.

1005—Panel strips and critical design sections

(*a*) A flat slab panel shall be considered as consisting of strips in each direction as follows:

A middle strip one half panel in width, symmetrical about panel center line and extending through the panel in the direction of the span for bending.

A column strip consisting of the two adjacent quarter-panels either side of the column center lines.

(*b*) The critical sections for bending are located as follows:

Sections for negative bending shall be taken along the edges of the panel, on column center lines between capitals and around the perimeters of column capitals.

Sections for positive bending shall be taken at mid-span of the strips.

(*c*) Only the reinforcement which crosses a critical section within a strip may be considered effective to resist bending in the strip at that section. Reinforcement which crosses such section at an angle with the center-line of the strip shall be assumed to contribute to the resistance of bending only its effective area in the direction of the strip, as defined in Section 104.

1006—Slab thickness and drop panels

(*a*) The thickness of a flat slab and the size and thickness of the drop panel, where used, shall be such that the compressive stress due to bending at the critical sections of any strip and the shear about the column capital and the drop panel shall not exceed the unit stresses allowed in concrete of the quality used.

(*b*) The shearing stresses in the slab outside the capital or drop panel shall be computed as provided in Section 807.

(*c*) Slab thickness shall not, however, be less than

$$\frac{L}{40} \text{ with drop panels}$$

or

$$\frac{L}{36} \text{ without drop panels}$$

(*d*) The thickness of the drop panel below the slab shall not be more than one-fourth the distance from the edge of the column capital to the edge of the drop panel.

1007—Capitals and brackets

(*a*) Where a column is without a flaring concrete capital the distance *c* shall be taken as the diameter of the column. Structural metal embedded in the slab or drop panel may be regarded as contributing to resistance in bending and shear.

(*b*) Where a reinforced concrete beam frames into a column without capital or bracket on the same side with the beam, the value of *c* may be taken as the width of the column plus twice the projection of the beam above or below the slab or drop panel for computing bending in strips parallel to the beam.

(*c*) Brackets capable of transmitting the negative bending and the shear in the column strips to the columns without excessive unit stress may be substituted for column capitals at exterior columns. The value of *c* where brackets are used shall be taken as twice the distance from the center of the column to a point where the bracket is 1½ inches thick, but not more than the thickness of the column plus twice the depth of the bracket.

(*d*) The average of the diameters *c* of the column capitals at the four corners of a panel shall be used in determining the bending in the middle strips of the panel. The average of the diameters *c* of the two column capitals at the ends of a column strip shall be used in determining bending in the strip.

1008—Arrangement of reinforcement

(*a*) Slab reinforcement shall be provided to resist the bending and bond stresses not only at critical sections, but also at intermediate sections.

(*b*) Bars shall be spaced evenly across strips or bands and the spacing shall not exceed three times the slab thickness.

(*c*) In exterior panels the reinforcement perpendicular to the discontinuous edge for positive bending, shall extend to the edge and have embedment of at least six inches in spandrel beams, walls or columns. All such reinforcement for negative bending shall be bent, hooked or otherwise anchored in spandrel beams, walls or columns.

1009—Openings in flat slabs

Openings of any size may be cut through a flat slab if provision is made for the total positive and negative resisting moments, as required in Sections 1002 or 1003, without exceeding the allowable stresses as given in Sections 305 and 306.

CHAPTER 11—REINFORCED CONCRETE COLUMNS AND WALLS

1100—Notation

A_c = Area of core of a spirally reinforced column measured to the outside diameter of the spiral; net area of concrete section of a composite column.

A_g = The overall or gross area of spirally reinforced or tied columns; the total area of the concrete encasement of combination columns.

A_r = Area of the steel or cast-iron core of a composite column; the area of the steel core in a combination column.

A_s = Effective cross-sectional area of reinforcement in compression in columns.

C = Ratio of allowable concrete stress, f_a, in axially loaded column to allowable fiber stress for concrete in flexure.

$D = \frac{t^2}{2R^2}$ = a factor, usually varying from 3 to 9. (The term R as used here is the radius of gyration of the entire column section.)

d = The least lateral dimension of a concrete column.

e = Eccentricity of the resultant load on a column, measured from the gravity axis.

$F = \frac{\textit{Yield point of pipe}}{45{,}000}$ (See Section 1106(b)).

f_a = Average allowable stress in the concrete of an axially loaded reinforced concrete column.

f_c = Computed concrete fiber stress in an eccentrically loaded column.

f'_c = Compressive strength of concrete at age of 28 days, unless otherwise specified.

f_p = Maximum allowable concrete fiber stress in an eccentrically loaded column.

f_r = Allowable unit stress in the metal core of a composite column.

f'_r = Allowable unit stress on unencased steel columns and pipe columns.

f_s = Nominal allowable stress in vertical column reinforcement.

f'_s = Useful limit stress of spiral reinforcement.

h = Unsupported length of column.

K = Least radius of gyration of a metal pipe section (in pipe columns).

$n = \dfrac{30{,}000}{f'_c}$

N = Axial load applied to reinforced concrete column.

p' = Ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals) of a spirally reinforced concrete column.

p_g = Ratio of the effective cross-sectional area of vertical reinforcement to the gross area A_g.

P = Total allowable axial load on a column whose length does not exceed ten times its least cross-sectional dimension.

P' = Total allowable axial load on a long column.

R = Least radius of gyration of a section.

t = Overall depth of column section.

1101—Limiting dimensions

(*a*) The following sections on reinforced concrete and composite columns, except Section 1107(a), apply to a short column for which the unsupported length is not greater than ten times the least dimension. When the unsupported length exceeds this value, the design shall be modified as shown in Section 1107(a). Principal columns in buildings shall have a minimum diameter of twelve inches, or in the case of rectangular columns, a minimum thickness of ten inches, and a minimum gross area of 120 sq in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of six inches.

1102—Unsupported length of columns

(*a*) For purposes of determining the limiting dimensions of columns, the unsupported length of reinforced concrete columns shall be taken as the clear distance between floor slabs, except that

1. In flat slab construction, it shall be the clear distance between the floor and the lower extremity of the capital, the drop panel or the slab, whichever is least.

2. In beam and slab construction, it shall be the clear distance between the floor and the under side of the deeper beam framing into the column in each direction at the next higher floor level.

3. In columns restrained laterally by struts, it shall be the clear distance between consecutive struts in each vertical plane; provided that to be an adequate support, two such struts shall meet the column at approximately the same level, and the angle between vertical planes through the struts shall not vary more than 15 degrees from a right angle. Such struts shall be of adequate dimensions and anchorage to restrain the column against lateral deflection.

4. In columns restrained laterally by struts or beams, with brackets used at the junction, it shall be the clear distance between the floor and the lower edge of the bracket, provided that the bracket width equals that of the beam or strut and is at least half that of the column.

(*b*) For rectangular columns, that length shall be considered which produces the greatest ratio of length to depth of section.

1103—Spirally reinforced columns

(*a*) *Allowable load*—The maximum allowable axial load, P, on columns with closely spaced spirals enclosing a circular concrete core reinforced with vertical bars shall be that given by formula (20).

$$P = A_g (0.225 f'_c + f_s p_g) \quad (20)$$

Wherein

A_g = the gross area of the column
f'_c = compressive strength of the concrete
f_s = nominal allowable stress in vertical column reinforcement, to be taken at forty percent of the minimum specification value of the yield point; viz., 16,000 psi for intermediate grade steel and 20,000 psi for rail or hard grade steel.*
p_g = ratio of the effective cross-sectional area of vertical reinforcement to the gross area, A_g.

(*b*) *Vertical reinforcement*—The ratio p_g shall not be less than 0.01 nor more than 0.08. The minimum number of bars shall be six, and the minimum diameter shall be ⅝ in. The center to center spacing of bars within the periphery of the column core shall not be less than 2½ times the diameter for round bars or three times the side dimension for square bars. The clear spacing between bars shall not be less than 1½ inches or 1½ times the maximum size of the coarse aggregate used. These spacing rules also apply to adjacent pairs of bars at a lapped splice; each pair of lapped bars forming a splice may be in contact, but the minimum clear spacing between one splice and the adjacent splice should be that specified for adjacent single bars.

(*c*) *Splices in vertical reinforcement*—Where lapped splices in the column verticals are used, the minimum amount of lap shall be as follows:

1. For deformed bars with concrete having a strength of 3000 psi or above, 20 diameters of bar of intermediate or hard grade steel. For bars of higher yield point, the amount of lap shall be

*Nominal allowable stresses for reinforcement of higher yield point may be established at forty percent of the yield point stress, but not more than 30,000 psi., when the properties of such reinforcing steels have been definitely specified by standards of A.S.T.M. designation. If this is done, the lengths of splice required by Section 1103 (c) shall be increased accordingly.

increased one diameter for each 1000 psi by which the allowable stress exceeds 20,000 psi. When the concrete strengths are less than 3000 psi, the amount of lap shall be one-third greater than the values given above.

2. For plain bars—the minimum amount of lap shall be twice that specified for deformed bars.

3. Welded splices or other positive connections may be used instead of lapped splices. Welded splices shall preferably be used in cases where the bar diameter exceeds 1½ in. An approved welded splice shall be defined as one in which the bars are butted and welded and that will develop in tension at least the yield point stress of the reinforcing steel used.

4. Where longitudinal bars are offset at a splice, the slope of the inclined portion of the bar with the axis of the column shall not exceed 1 in 6, and the portions of the bar above and below the offset shall be parallel to the axis of the column. Adequate horizontal support at the offset bends shall be treated as a matter of design, and may be provided by metal ties, spirals or parts of the floor construction. Metal ties or spirals so designed shall be placed near (never more than 8 bar diameters from) the point of bend. The horizontal thrust to be resisted may be assumed as 1½ times the horizontal component of the nominal stress in the inclined portion of the bar.

Offset bars shall be bent before they are placed in the forms. No field bending of bars partially embedded in concrete shall be permitted.

(*d*) *Spiral reinforcement*—The ratio of spiral reinforcement, p', shall not be less than the value given by formula (21).

$$p' = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f'_s} \qquad (21)$$

Wherein

p' = ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals).

f'_s = useful limit stress of spiral reinforcement, to be taken as 40,000 psi for hot rolled rods of intermediate grade, 50,000 psi for rods of hard grade, and 60,000 psi for cold drawn wire.

The spiral reinforcement shall consist of evenly spaced continuous spirals held firmly in place and true to line by vertical spacers, using at least two for spirals 20 in. or less in diameter, three for spirals 20 to 30 in. in diameter and four for spirals more than 30 in. in diameter or composed of

spiral rods ⅝-in. or larger in size. The spirals shall be of such size and so assembled as to permit handling and placing without being distorted from the designed dimensions. The material used in spirals shall have a minimum diameter of ¼ in. for rolled bars or No. 4 W. & M. gage for drawn wire. Anchorage of spiral reinforcement shall be provided by 1½ extra turns of spiral rod or wire at each end of the spiral unit. Splices, when necessary shall be made in spiral rod or wire by welding or by a lap of 1½ turns. The center to center spacing of the spirals shall not exceed one-sixth of the core diameter. The clear spacing between spirals shall not exceed 3 in. nor be less than 1⅜ in. or 1½ times the maximum size of coarse aggregate used. The reinforcing spiral shall extend from the floor level in any story or from the top of the footing in the basement, to the level of the lowest horizontal reinforcement in the slab, drop panel or beam above. In a column with a capital, it shall extend to a plane at which the diameter or width of the capital is twice that of the column.

(*e*) *Protection of reinforcement*—The column reinforcement shall be protected everywhere by a covering of concrete cast monolithically with the core, for which the thickness shall not be less than 1½ in. nor less than 1½ times the maximum size of the coarse aggregate, nor shall it be less than required by the fire protection and weathering provisions of Section 507.

(*f*) *Isolated column with multiple spirals*—In case two or more interlocking spirals are used in a column, the outer boundary of the column shall be taken as a rectangle the sides of which are outside the extreme limits of the spiral at a distance equal to the requirements of Section 1103(*e*).

(*g*) *Limits of section of column built monolithically with wall*—For a spiral column built monolithically with a concrete wall or pier, the outer boundary of the column section shall be taken either as a circle at least 1½ in. outside the column spiral or as a square or rectangle of which the sides are at least 1½ in. outside the spiral or spirals.

(*h*) *Equivalent circular columns*—As an exception to the general procedure of utilizing the full gross area of the column section, it shall be permissible to design a circular column and to build it with a square, octagonal, or other shaped section of the same least lateral dimension. In such case, the allowable load, the gross area considered, and the required percentages of reinforcement shall be taken as those of the circular column.

1104—Tied columns

(*a*) *Allowable load*—The maximum allowable axial load on columns reinforced with longitudinal bars and separate lateral ties shall be 80 per cent of that given by Formula (20). The ratio, p_g, to be considered

in tied columns shall not be less than 0.01 nor more than 0.04. The longitudinal reinforcement shall consist of at least four bars, of minimum diameter of ⅝ inch. Splices in reinforcing bars shall be made as described in Section 1103 (*c*). The spacing requirements for vertical reinforcement in Section 1103 (b) shall also apply for all tied columns.

(*b*) *Lateral ties*—Lateral ties shall be at least ¼ in. in diameter and shall be spaced apart not over 16 bar diameters, 48 tie diameters or the least dimension of the column. When there are more than four vertical bars, additional ties shall be provided so that every longitudinal bar is held firmly in its designed position and has lateral support equivalent to that provided by a 90-degree corner of a tie.

(*c*) *Limits of column section*—In a tied column which for architectural reasons has a larger cross section than required by considerations of loading, a reduced effective area, A_g, not less than one-half of the total area may be used in applying the provisions of Section 1104 (a).

1105—Composite columns

(*a*) *Allowable load*—The allowable load on a composite column, consisting of a structural steel or cast-iron column thoroughly encased in concrete reinforced with both longitudinal and spiral reinforcement, shall not exceed that given by formula (22).

$$P = 0.225 A_c f'_c + f_s A_s + f_r A_r \quad (22)$$

Wherein A_c = net area of concrete section
$= A_g - A_s - A_r$
A_s = cross-sectional area of longitudinal bar reinforcement.
A_r = cross-sectional area of the steel or cast-iron core.
f_r = allowable unit stress in metal core, not to exceed 16,000 psi for a steel core; or 10,000 psi for a cast-iron core.

The remaining notation is that of Section 1103.

(*b*) *Details of metal core and reinforcement*—The cross-sectional area of the metal core shall not exceed 20 percent of the gross area of the column. If a hollow metal core is used it shall be filled with concrete. The amounts of longitudinal and spiral reinforcement and the requirements as to spacing of bars, details of splices and thickness of protective shell outside the spiral shall conform to the limiting values specified in Section 1103 (b), (c), (d) and (e). A clearance of at least three inches shall be maintained between the spiral and the metal core at all points except that when the core consists of a structural steel H-column, the minimum clearance may be reduced to two inches.

(*c*) *Splices and connections of metal cores*—Metal cores in composite columns shall be accurately milled at splices and positive provision shall be made for alignment of one core above another. At the column base,

provision shall be made to transfer the load to the footing at safe unit stresses in accordance with Section 305 (a). The base of the metal section shall be designed to transfer the load from the entire composite column to the footing, or it may be designed to transfer the load from the metal section only, provided it is so placed in the pier or pedestal as to leave ample section of concrete above the base for the transfer of load from the reinforced concrete section of the column by means of bond on the vertical reinforcement and by direct compression on the concrete. Transfer of loads to the metal core shall be provided for by the use of bearing members such as billets, brackets or other positive connections; these shall be provided at the top of the metal core and at intermediate floor levels where required. The column as a whole shall satisfy the requirements of formula (22) at any point; in addition to this, the reinforced concrete portion shall be designed to carry, in accordance with formula (20), all floor loads brought onto the column at levels between the metal brackets or connections. In applying formula (20), the value of A_g shall be interpreted as the area of the concrete section outside the metal core, and the allowable load on the reinforced concrete section shall be further limited to $0.35\,f'_c A_g$. Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders.

(*d*) *Allowable load on metal core only*—The metal cores of composite columns shall be designed to carry safely any construction or other loads to be placed upon them prior to their encasement in concrete.

1106—Combination columns

(*a*) *Steel columns encased in concrete*—The allowable load on a structural steel column which is encased in concrete at least 2½ inches thick over all metal (except rivet heads) reinforced as hereinafter specified, shall be computed by formula (23).

$$P = A_r f'_r \left[1 + \frac{A_g}{100\,A_r}\right] \qquad (23)$$

Wherein A_r = cross-sectional area of steel column.
f'_r = allowable stress for unencased steel column.
A_g = total area of concrete section.

The concrete used shall develop a compressive strength, f'_c, of at least 2000 psi at 28 days. The concrete shall be reinforced by the equivalent of welded wire mesh having wires of No. 10 W. and M. gage, the wires encircling the column being spaced not more than four inches apart and those parallel to the column axis not more than eight inches apart. This mesh shall extend entirely around the column at a distance of one inch inside the outer concrete surface and shall be lap-spliced at least forty wire diameters and wired at the splice. Special brackets shall be used

to receive the entire floor load at each floor level. The steel column shall be designed to carry safely any construction or other loads to be placed upon it prior to its encasement in concrete.

(*b*) *Pipe columns*—The allowable load on columns consisting of steel pipe filled with concrete shall be determined by formula (24).

$$P = 0.225 f'_c A_c + f'_r A_r \quad \text{.....(24)}$$

The value of f'_r shall be given by formula (25).

$$f'_r = \left[18{,}000 - 70\frac{h}{K}\right]F \quad \text{.....(25)}$$

Wherein f'_r = allowable unit stress in metal pipe.
h = unsupported length of column
K = least radius of gyration of metal pipe section.

$$F = \frac{\textit{yield point of pipe.}}{45{,}000}$$

If the yield point of the pipe is not known, the factor F shall be taken as 0.5.

1107—Long columns

(*a*) The maximum allowable load, P', on axially loaded reinforced concrete or composite columns having an unsupported length, h, greater than ten times the least lateral dimension, d, shall be given by formula (26).

$$P' = P\left[1.3 - .03\frac{h}{d}\right] \quad \text{.....(26)}$$

where P is the allowable axial load on a short column as given by sections 1103, 1104 and 1105.

The maximum allowable load, P', on eccentrically loaded columns in which $\frac{h}{d}$ exceeds ten shall also be given by formula (26), in which P is the allowable eccentrically applied load on a short column as determined by the provisions of Sections 1109 and 1110. In long columns subjected to definite bending stresses, as determined in Section 1108, the ratio $\frac{h}{d}$ shall not exceed twenty.

1108—Bending moments in columns

(*a*) The bending moments in the columns of all reinforced concrete structures shall be determined on the basis of loading conditions and restraint and shall be provided for in the design. When the stiffness and strength of the columns are utilized to reduce moments in beams, girders,

or slabs, as in the case of rigid frames, or in other forms of continuous construction wherein column moments are unavoidable, they shall be provided for in the design. In building frames, particular attention shall be given to the effect of unbalanced floor loads on both exterior and interior columns and of eccentric loading due to other causes. In computing moments in columns, the far ends may be considered fixed. Columns shall be designed to resist the axial forces from loads on all floors, plus the maximum bending due to loads on a single adjacent span of the floor under consideration.

Resistance to bending moments at any floor level shall be provided by distributing the moment between the columns immediately above and below the given floor in proportion to their relative stiffnesses and conditions of restraint.

1109—Determination of combined axial and bending stresses

(*a*) In a reinforced concrete column, designed by the methods of this Chapter, which is (1) symmetrical about two perpendicular planes through its axis and (2) subject to an axial load, N, combined with bending in one or both of the planes of symmetry (but with the ratio of eccentricity to depth, e/t, no greater than 1.0 in either plane), the combined fiber stress in compression may be computed on the basis of recognized theory applying to uncracked sections, using formula (27).

$$f_c = \frac{N}{A_g}\left[\frac{1 + \frac{De}{t}}{1 + (n - 1)p_g}\right] \quad \ldots\ldots (27)$$

Equating this calculated stress, f_c, to the allowable stress, f_p, in formula (29) it follows that the column can be designed for an equivalent axial load, P, as given by formula (28).*

$$P = N\left[1 + \frac{CDe}{t}\right] \quad \ldots\ldots (28)$$

When bending exists on both axes of symmetry, the quantity $\frac{De}{t}$ is to be computed as the numerical sum of the $\frac{De}{t}$ quantities in the two directions.

(*b*) For columns in which the load, N, has an eccentricity, e, greater than the column depth, t, or for beams subject to small axial loads, the determination of the fiber stress f_c shall be made by use of recognized theory for cracked sections, based on the assumption that no ten-

*For approximate or trial computations, D may be taken as eight for a circular spiral column and as five for a rectangular tied or spiral column.

sion exists in the concrete. For such cases the tensile steel stress shall also be investigated.

1110—Allowable combined axial and bending stress

(*a*) For spiral and tied columns, eccentrically loaded or otherwise subjected to combined axial compression and flexural stress, the maximum allowable compressive stress, f_p, is given by formula (29).

$$f_p = f_a \left[\frac{1 + \frac{De}{t}}{1 + \frac{CDe}{t}} \right] = f_a \left[\frac{t + De}{t + CDe} \right] \quad \text{.....(29)}$$

Wherein the notation is that of Section 1103 and 1109, and, in addition f_a is the average allowable stress in the concrete of an axially loaded reinforced concrete column, and C is the ratio of f_a to the allowable fiber stress for members in flexure. Thus $f_a = \frac{0.225\, f'_c + f_s p_g}{1 + (n - 1)p_g}$ for spiral columns and 0.8 of this value for tied columns, and $C = \frac{f_a}{0.45\, f'_c}$.

(*b*) For tied columns which are designed to withstand combined axial and bending stresses, the limiting total steel ratio of 0.04 prescribed in Section 1104 may be increased to 0.08, provided that the amount of steel spliced by lapping shall not exceed a steel ratio of 0.04 in any 3-ft. length of column. The size of the column designed under this provision shall in no case be less than that required to withstand axial load alone in accordance with Section 1104.

1111—Wind stresses

(*a*) When the allowable stress in columns is modified to provide for combined axial load and bending, and the stress due to wind loads is also added, the total shall still come within the allowable values specified for wind loads in Section 603 (c).

1112—Reinforced concrete walls

(*a*) The allowable stresses in reinforced concrete bearing walls with minimum reinforcement as required by Section 1112(i), shall be $0.25f'_c$ for walls having a ratio of height to thickness of ten or less, and shall be reduced proportionally to $0.15f'_c$ for walls having a ratio of height to thickness of twenty-five. When the reinforcement in bearing walls is designed, placed and anchored in position as for tied columns, the allowable stresses shall be on the basis of Section 1104, as for columns. In the case of concentrated loads, the length of the wall to be considered as effective for each shall not exceed the center to center distance between

loads, nor shall it exceed the width of the bearing plus four times the wall thickness. The ratio p_s shall not exceed 0.04.

(*b*) Walls shall be designed for any lateral or other pressure to which they are subjected. Proper provision shall be made for eccentric loads and wind stresses. In such designs the allowable stresses shall be as given in Section 305(a) and 603(c).

(*c*) Panel and enclosure walls of reinforced concrete shall have a thickness of not less than five inches and not less than one thirtieth the distance between the supporting or enclosing members.

(*d*) Bearing walls of reinforced concrete in building of fire-resistive construction shall be not less than six inches in thickness for the uppermost fifteen feet of their height; and for each successive twenty-five feet downward, or fraction thereof, the minimum thickness shall be increased one inch. In two-story dwellings the walls may be six inches in thickness throughout.

(*e*) In buildings of non-fire-resistive construction bearing walls of reinforced concrete shall not be less than one and one-third times the thickness required for buildings of fire-resistive construction, except that for dwellings of two stories or less in height the thickness of walls may be the same as specified for buildings of fire-resistive construction.

(*f*) Exterior basement walls, foundation walls, fire walls and party walls shall not be less than eight inches thick whether reinforced or not.

(*g*) Reinforced concrete bearing walls shall have a thickness of at least one twenty-fifth of the unsupported height or width, whichever is the shorter; provided however, that approved buttresses, built-in columns, or piers designed to carry all the vertical loads, may be used in lieu of increased thickness.

(*h*) Reinforced concrete walls shall be anchored to the floors, columns, pilasters, buttresses and intersecting walls with reinforcement at least equivalent to three-eighths inch round bars twelve inches on centers, for each layer of wall reinforcement.

(*i*) Reinforced concrete walls shall be reinforced with an area of steel in each direction, both vertical and horizontal, at least equal to 0.0025 times the cross-sectional area of the wall, if of bars, and 0.0018 times the area if of electrically welded wire fabric.* The wire of the welded fabric shall be of not less than No. 10 W. & M. gage. Walls more than ten inches in thickness shall have the reinforcement for each direction placed in two layers parallel with the faces of the wall. One layer consisting of not less than one-half and not more than two-thirds the total required shall be placed not less than two inches nor more than one-third the thickness of the wall from the exterior surface. The other layer, comprising the balance of the required reinforcement, shall be placed not

less than three-fourths inches and not more than one-third the thickness of the wall from the interior surface. Bars, if used, shall not be less than the equivalent of three-eighths inch round bars, nor shall they be spaced more than eighteen inches on centers. Welded wire* reinforcement for walls shall be in flat sheet form.

(*j*) In addition to the minimum as prescribed in 1112(i) there shall be not less than two five-eighths inch diameter bars around all window or door openings. Such bars shall extend at least twenty-four inches beyond the corner of the openings.

(*k*) Where reinforced concrete bearing walls consist of studs or ribs tied together by reinforced concrete members at each floor level, the studs may be considered as columns, but the restrictions as to minimum diameter or thickness of columns shall not apply.

*Expanded metal has been omitted until a specification can be formulated.

CHAPTER 12—FOOTINGS

1201—Scope

(*a*) The requirements prescribed in Sections 1202 to 1209 apply only to isolated footings.*

1202—Loads and reactions

(*a*) Footings shall be proportioned to sustain the applied loads and induced reactions without exceeding the allowable stresses as prescribed in Sections 305 and 306, and as further provided in Sections 1205, 1206 and 1207.

(*b*) In cases where the footing is concentrically loaded and the member being supported does not transmit any moment to the footing, computations for moments and shears shall be based on an upward reaction assumed to be uniformly distributed per unit area or per pile and a downward applied load assumed to be uniformly distributed over the area of the footing covered by the column, pedestal, wall, or metallic column base.

(*c*) In cases where the footing is eccentrically loaded and/or the member being supported transmits a moment to the footing, proper allowance shall be made for any variation that may exist in the intensities of reaction and applied load consistent with the magnitude of the applied load and the amount of its actual or virtual eccentricity.

(*d*) In the case of footings on piles, computations for moments and shears may be based on the assumption that the reaction from any pile is concentrated at the center of the pile.

1203—Sloped or stepped footings

(*a*) In sloped or stepped footings, the angle of slope or depth and location of steps shall be such that the allowable stresses are not exceeded at any section.

(*b*) In sloped or stepped footings, the effective cross-section in compression shall be limited by the area above the neutral plane.

(*c*) Sloped or stepped footings shall be cast as a unit.

1204—Bending moment

(*a*) The external moment on any section shall be determined by passing through the section a vertical plane which extends completely across the footing, and computing the moment of the forces acting over the entire area of the footing on one side of said plane.

(*b*) The greatest bending moment to be used in the design of an isolated footing shall be the moment computed in the manner prescribed in Section 1204 (a) at sections located as follows:

*The committee is not prepared at this time to make recommendations for combined footings—those supporting more than one column or wall.

1. At the face of the column, pedestal or wall, for footings supporting a concrete column, pedestal or wall.

2. Halfway between the middle and the edge of the wall, for footings under masonry walls.

3. Halfway between the face of the column or pedestal and the edge of the metallic base, for footings under metallic bases.

(*c*) The width resisting compression at any section shall be assumed as the entire width of the top of the footing at the section under consideration.

(*d*) In one-way reinforced footings, the total tensile reinforcement at any section shall provide a moment of resistance at least equal to the moment computed in the manner prescribed in Section 1204(a); and the reinforcement thus determined shall be distributed uniformly across the full width of the section.

(*e*) In two-way reinforced footings, the total tensile reinforcement at any section shall provide a moment of resistance at least equal to eighty-five percent of the moment computed in the manner prescribed in Section 1204(a); and the total reinforcement thus determined shall be distributed across the corresponding resisting section in the manner prescribed for square footings in Section 1204(f), and for rectangular footings in Sec. 1204(g).

(*f*) In two-way square footings, the reinforcement extending in each direction shall be distributed uniformly across the full width of the footing.

(*g*) In two-way rectangular footings, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing. In the case of the reinforcement in the short direction, that portion determined by formula (30) shall be uniformly distributed across a band-width (B) centered with respect to the center line of the column or pedestal and having a width equal to the length of the short side of the footing. The remainder of the reinforcement shall be uniformly distributed in the outer portions of the footing.

$$\frac{\textit{Reinforcement in band-width } (B)}{\textit{Total reinforcement in short direction}} = \frac{2}{(S + 1)} \quad \ldots\ldots\ldots (30)$$

In formula (30), "S" is the ratio of the long side to the short side of the footing.

1205—Shear and bond

(*a*) The critical section for shear to be used as a measure of diagonal tension shall be assumed as a vertical section obtained by passing a series of vertical planes through the footing, each of which is parallel to a corresponding face of the column, pedestal, or wall and located a

distance therefrom equal to the depth d for footings on soil, and one-half the depth d for footings on piles.

(*b*) Each face of the critical section as defined in Section 1205(a) shall be considered as resisting an external shear equal to the load on an area bounded by said face of the critical section for shear, two diagonal lines drawn from the column or pedestal corners and making 45° angles with the principal axes of the footing, and that portion of the corresponding edge or edges of the footing intercepted between the two diagonals.

(*c*) Critical sections for bond shall be assumed at the same planes as those prescribed for bending moment in Section 1204(b); also at all other vertical planes where changes of section or of reinforcement occur.

(*d*) Computations for shear to be used as a measure of bond shall be based on the same section and loading as prescribed for bending moment in Section 1204(a).

(*e*) The total tensile reinforcement at any section shall provide a bond resistance at least equal to the bond requirement as computed from the following percentages of the external shear at the section:

1. In one-way reinforced footings, 100 percent.
2. In two-way reinforced footings, 85 percent.

(*f*) In computing the external shear on any section through a footing supported on piles, the entire reaction from any pile whose center is located six inches or more outside the section shall be assumed as producing shear on the section; the reaction from any pile whose center is located six inches or more inside the section shall be assumed as producing no shear on the section. For intermediate positions of the pile center, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight-line interpolation between full value at six inches outside the section and zero value at six inches inside the section.

(*g*) For allowable shearing stresses, see Section 305 and 808.

(*h*) For allowable bond stresses, see Section 305 and 901 to 905.

1206—Transfer of stress at base of column

(*a*) The stress in the longitudinal reinforcement of a column or pedestal shall be transferred to its supporting pedestal or footing either by extending the longitudinal bars into the supporting member, or by dowels.

(*b*) In case the transfer of stress in the reinforcement is accomplished by extension of the longitudinal bars, they shall extend into the sup-

porting member the distance required to transfer to the concrete, by allowable bond stress, their full working value.

(*c*) In cases where dowels are used, their total sectional area shall be not less than the sectional area of the longitudinal reinforcement in the member from which the stress is being transferred. In no case shall the number of dowels per member be less than four and the diameter of the dowels shall not exceed the diameter of the column bars by more than one-eighth inch.

(*d*) Dowels shall extend up into the column or pedestal a distance at least equal to that required for lap of longitudinal column bars (see Section 1103) and down into the supporting pedestal or footing the distance required to transfer to the concrete, by allowable bond stress, the full working value of the dowel. (See Section 906 (*d*).)

(*e*) The compressive stress in the concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting pedestal or footing. The unit compressive stress on the loaded area shall not exceed the bearing stress allowable for the quality of concrete in the supporting member as limited by the ratio of the loaded area to the supporting area.

(*f*) For allowable bearing stresses see Table 305(a), Section 305.

(*g*) In sloped or stepped footings, the supporting area for bearing may be taken as the top horizontal surface of the footing, or assumed as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the area actually loaded, and having side slopes of one vertical to two horizontal.

1207—Pedestals and footings (plain concrete)

(*a*) The allowable compressive unit stress on the gross area of a concentrically loaded pedestal shall not exceed $0.25f'_c$. Where this stress is exceeded, reinforcement shall be provided and the member designed as a reinforced concrete column.

(*b*) The depth and width of a pedestal or footing of plain concrete shall be such that the tension in the concrete shall not exceed $.03f'_c$, and the average shearing stress shall not exceed $.02f'_c$ taken on sections as prescribed in Section 1204 and 1205 for reinforced concrete footings.

1208—Footings supporting round columns

(*a*) In computing the stresses in footings which support a round or octagonal concrete column or pedestal, the "face" of the column or pedestal shall be taken as the side of a square having an area equal to the area enclosed within the perimeter of the column or pedestal.

1209—Minimum edge-thickness

(*a*) In reinforced concrete footings, the thickness above the reinforcement at the edge shall be not less than six in. for footings on soil, nor less than twelve in. for footings on piles.

(*b*) In plain concrete footings, the thickness at the edge shall be not less than eight in. for footings on soil, nor less than fourteen in. above the tops of the piles for footings on piles.

Title No. 47-44

Thin Concrete Topping Restores Old Pavement*

By H. WALTER HUGHES†

SYNOPSIS

Lack of information on bonding new concrete to old led to laboratory bonding tests on wet and dry surfaces. Favorable results led to experiments on topping concrete road slabs, which required special techniques in bonding, screeding, compacting and finishing. No sign of surface disintegration has been observed after 6 years on any of the sections. Results indicate that the use of low water-cement ratio, graded mixes and compaction gives ample bond for thin toppings of floors or badly scaled pavements in which the concrete is sound. Concrete pavements containing low durability aggregates can also be given durable wearing surfaces with thin high-grade toppings.

INTRODUCTION

The use of salt (sodium chloride) in combating ice and snow on pavements and sidewalks creates a need to maintain durable wearing surfaces. Rather than rebuild entire slabs, most cities resurface the scaled portions. In Rochester, N. Y., sidewalks failed in one or two winters as a result of salt action. Such failure follows the "usual method" of resurfacing. "Usual" was to place a 2- to 4-in. slump concrete by hand on the old surface which had been cleaned with air and water and well grouted after all loose particles had been removed.

ACI Committee 804 had investigated similar problems in the construction of wearing surfaces for heavy duty floors. Methods and techniques were discussed, culminating in the 1938 report.‡ There was still a lack of test data on the bond between the structural slab and topping applied after the slab had hardened.

BOND TESTS

Various methods of bonding were studied; and apparatus was developed to obtain a value of the bond strength in shear (Fig. 1). It consisted of a piece of tubing about ⅛ in. larger in diameter than the core to be drilled from the test slab in which the core was placed and aligned, with the bond line between topping and slab at one end of the tube. Once in position, the core was held firmly by filling the remaining space in the tube with sulfur cement. The assembly was then placed in a jig and held securely with a

*Presented at the ACI 47th Annual Convention, San Francisco, Calif., Feb. 21, 1951. Title No. 47-44 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 8, Apr. 1951, *Proceedings* V. 47. Separate prints are available at 35 cents each. **Discussion** (copies in triplicate) should reach the Institute not later than Aug. 1, 1951. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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‡Committee 804, "Concrete Wearing Surfaces for Floors," ACI JOURNAL, Sept. 1938, *Proc.* V. 35, pp. 21-32.

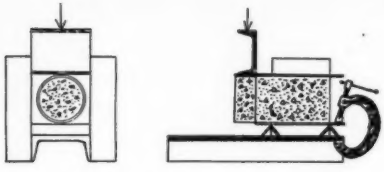

Fig. 1—Jig for testing bond between base and topping.

C-clamp. The jig was placed in the testing machine and load applied to the topping which protruded from the tube.

To compare the bond strength of various cementing materials when the topping was placed at low water content and finished with a compactor power float, a 5 ft x 10 ft x 6 in. slab was constructed. After curing and aging, the slab was prepared for topping by picking.

Four cementing materials were tested. White cement had been recommended as proving superior in use. (The only characteristic that appeared to have an influence was its fineness.) High early strength cement and mortar cement were chosen because of their fineness, and Vinsol resin treated Type II cement because of workability.

Three methods of application were used with each cementing material.

Method A—Dusting dry cement over the dry slab, followed by covering the area with a very wet 1:1 cement and sand mortar.

Method B—The slab was wetted and dry cement dusted over the surface, followed by a creamy mortar.

Method C—The slab was wetted, allowed to absorb water until absorption was nearly satisfied, and a creamy 1:1 mortar was worked in with a stiff brush.

The bonding materials were placed just prior to the topping mixture. After the topping was placed, it was hand tamped, screeded and finished by a compactor power float. The slab was cured under wet burlap for ten days and then allowed to dry. After about three months, cores were taken from the various sections and tested for strength in shear at the bond line.

Mix proportions

Topping proportions were the result of laboratory tests. Crushed stone and sand were combined to get the smallest percentage of loose volume voids. The stone was graded ½ to ⅜ in. and ⅜ to ⅛ in. The sand had a fineness modulus of 2.80. Seven bags of cement per cu yd were used and mixing water held between 3 and 3½ gal. per sack of cement.

The percentage of fine aggregate varied between 30 and 40 percent depending upon the gradation of the coarse aggregate. A typical batch using 40 percent fine aggregate follows:

Material	*Quantity, lb*
Cement (7 bags)	658
Water (21 gal.)	175
Sand	1267
No. 1 stone (½–⅜-in.)	760
No. 1A stone (⅜–⅛-in.)	1140
	4000

Results

The results reported in Table 1 are the average of cores removed from each of the 12 sections. The type of slab surface appears to exert a major influence on the bond strength. The low result in Method B in both mortar and Vinsol resin treated cement sections was caused by aggregate being exposed in the surfaces of the slab. It was found from examination of the cores after testing that clean mortar, comparable to coarse sand paper, gives the highest strength in bond. The lowest values were obtained where coarse aggregate surface was about 50 percent of the area of the core. The core surfaces consisting largely of mortar broke back of the bond line.

TABLE 1—BOND STRENGTH IN SHEAR, psi

Method of bonding	White cement	High early cement	Mortar cement	Vinsol resin cement
A	220	255	262	318
B	327	242	136	266
C	340	303	312	504

Cores 3 in. diameter. Each result is the average of three cores.

About this time, the Vannier Co. had to use an aggregate of poor resistance to wear and weathering for a loading dock pavement at the Curtis-Wright factory. The problem was solved by using the aggregate in the slab and covering with a 1-in. topping of graded aggregate, surfaced with a compactor power float. Some of the topping was applied monolithically and some as a separate course.

Four of the cores were subjected to 194 cycles of continuous wet freezing and thawing to determine the durability of the bond. The bond of wearing course was unaffected in all four cores and the broom marks on the surface of three of the cores were as well defined as at the beginning of the test (Fig. 2). The question of bond durability should hardly arise when this type of wearing course is laid.

RESURFACING PAVEMENTS

The results of these laboratory tests led to field tests to determine the feasibility of resurfacing disintegrated concrete pavements where the bond would be subjected to extreme temperatures.

Pavements were resurfaced at four locations. Two were on arterial and two on residential streets. All were at or near intersections where they were subjected to heavy traffic and were kept clear during the winter with heavy applications of salt.

The thickness of the wearing course depended on the depth of the disintegrated concrete and varied between 1 and 2 in.

Due to the bad spalling of the pavement, the method recommended for floors appeared too costly and preparing the surface by hand pick method too time-consuming.

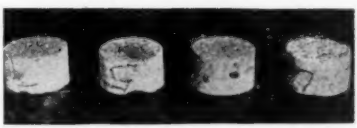

Fig. 2—Bond of wearing course to base was unaffected by 194 cycles of freezing and thawing.

Fig. 3—Dolly-mounted air drill preparing base for topping

Construction procedure

Good results were obtained by mounting an air drill, equipped with a 2¾-in. star bit, on a dolly (Fig. 3). Controls were arranged to limit the stroke of the drill. This allowed the bit to shatter and clean the surface without cracking the slab, the bit rebounding after the shattering blow. Continuous slow movement of the dolly created a smooth, even surface. The loose material was swept off the surface, the remaining dirt blown off with air and the surface washed by high pressure water. This washing also helped to saturate the slab. Later a three drill machine was developed to speed up the work (Fig. 4).

Laboratory experience had shown better bond strength with the minimum of coarse aggregate exposed at the bond line. The use of drills made it easier

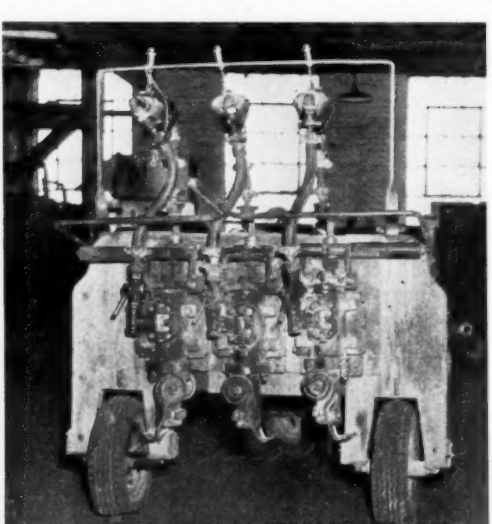

Fig. 4—Improved gang drill for speedy base preparation

Fig. 5—Dry topping mix placed on prepared and grouted base

to obtain this type of surface as well as giving a surface nearer a true plane to obtain uniform thickness.

After the prepared surface had been cleaned and washed, 1 x 12 in. boards were laid along the edges of the sections to be concreted. These were held in place by bags of sand. All free water was broomed off and the slab grouted with a 1:1 cement-sand creamy grout, well broomed.

The prepared and grouted surface and the consistency of the concrete used in resurfacing is shown in Fig. 5. After placing and hand tamping, the concrete was screeded to grade (Fig. 6). It was then surfaced and compacted with a compactor power float (Fig. 7), followed by the usual broom treatment standard in concrete pavement construction.

Fig. 6—Placing, tamping and screeding topping mix

Fig. 7—Using compactor power float on thin topping

The finished surface was covered with damp sand as soon as initial set took place. The following day the sand was saturated and kept wet for 7 days. No surface checking or cracking was observed on any of the resurfaced areas.

One area, a bus stop on Main Street, replaced an asphalt pavement and was 3 in. thick. This was placed in two layers and each layer rolled with a light roller. The usual procedure of screeding and finishing followed the rolling. The pavement base had been laid without expansion or contraction joints. The base had cracked to relieve these stresses; some cracks were wide, others were hardly noticeable. The wide cracks were carried through the new topping with premolded expansion joint material; the small openings were not. After the first winter the surface had the same crack pattern as the base.

During the next three years the edges of these cracks remained sharp and did not ravel or break back. No maintenance was done since it was desired to observe whether breakback and breaking at bond would occur at these points. After four years some of the broom marks were still visible on the surfaces under heavy traffic.

All these resurfaced areas were left exposed through four winters and two are still exposed. Fig. 8 shows the condition of a resurfaced area repaired in 1943.

No sign of surface disintegration has been observed on any of the sections. Some failure of bond was observed along some edges that were cast against

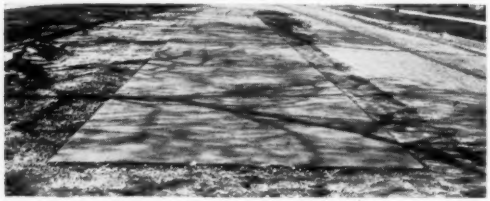

Fig. 8—Resurfaced area (center) after four winters. Compare with surrounding area repaired with another material

the 1 x 12 in. boards. The edges were well surfaced but the riding of one side of the machine on these boards apparently prevented the compactor hammers from driving the topping into the slab. This can be overcome by using adequate forms.

Costs

The following cost figures were turned in by the job foreman. For an area of 5512 sq ft the cost per sq ft was:

Preparation of surface	$0.09
Concrete resurfacing (material and labor)	0.10
Equipment rental	0.01
	$0.20

This figure is based on 1943 labor costs and concrete cost of $9.00 per cu yd. Another area of 3334 sq ft which involved a greater amount of disintegration cost:

Preparation of surface	$0.13
Concrete resurfacing (labor and material)	0.12
Equipment rental	0.01
	$0.26

SUMMARY

The results from laboratory and field tests indicate (1) that the use of low water-cement ratio, graded mixes and compaction gives sufficient bond for floor and pavement topping. (2) Badly scaled pavements in which the base concrete is sound can be resurfaced by this method, and (3) concrete pavements containing second grade aggregates with respect to durability can be made durable with high grade topping. (4) The question of bond durability should hardly arise when this type of wearing course is laid. (5) Resistance to the action of salt can be obtained by the methods of finishing used, including wet sand curing.

Experience has shown that dense, weather-resistant concrete surfaces can be produced by using dry, harsh mixtures, finished with the compactor power float.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Equivalent elastic systems in the analysis of continuous structures—II

L. A. Beaufoy and A. F. S. Diwan, *Concrete and Constructional Engineering* (London), V. 45, No. 12, Dec. 1950, pp. 427-434

Reviewed by Glenn Murphy

In this installment the authors show details of analysis of a four-span gable arch on flexible columns, including the development of an influence line for moment.

Piles 87 ft long at Plymouth

Concrete and Constructional Engineering (London) V. 46, No. 1, Jan. 1951, pp 28-30

Reviewed by Glenn Murphy

This article describes the construction and erection of thirty-three 21-in. square precast reinforced concrete piles used in construction of a wharf at Plymouth. Some of the piles which were 87 ft long when driven were subsequently lengthened to 107 ft.

Rectangular plate, elastically supported along its edges, under any condition of loading (La lastra rettangolare, sostenuta elaticamente lungo tutto il conforno, sottoposto a qualunque condizione di carico)

Piero Pozzati, *Giornale Del Genio Civile* (Rome), V. 88, No. 3, Mar. 1950, pp. 145-155

Reviewed by Gennaro Mianulli

This is the fourth of a series of articles on flat plates. The author states that it is impossible to analyze these conditions with the usual procedure followed in Part 1. The difficulty encountered is discussed thoroughly. To facilitate the design, tables and diagrams are worked out for bending moments along the fixed edges and for conditions of deformations occurring for yielding supports.

Hipped plate and shell roof construction in reinforced concrete

A. Krysztal, *Journal of the Institution of Engineers Australia* (Sydney), V. 22, No. 10-11, Oct.-Nov. 1950, pp. 235-245

Author's Summary

This paper describes a simple statical method of designing reinforced concrete hipped plate and cylindrical shell roofs. Dimensioned illustrations and recommended proportions of structures of these types are given and the paper concludes with an example of a hipped plate roof design.

Partitions function as columns

Paul Weidlinger, *Architectural Record*, V. 109, No. 1, Jan. 1951, pp. 134-139

A brief description of a system of multi-story construction in which reinforced concrete partition walls are the supporting elements, eliminating separate columns. Broken shaped partitions, especially suitable for apartments and two-way ribbed floors, work together to carry imposed loads. Irregular spacing imposes design complications which should be amenable to modern methods of experimental stress analysis.

A continuous prestressed concrete girder bridge in France

Concrete and Constructional Engineering (London), V. 46, No. 1, Jan. 1951, pp. 21-27

Reviewed by Glenn Murphy

Design and construction details of a 154-ft continuous prestressed concrete girder constructed in France are given. The bridge carries a double track railway, two roads,

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and two foot paths. Freyssinet type prestressing cables were used in the precast beams. The deck is a fully prestressed slab. A table of component and resultant stresses is included.

High frequency vibrator (Vibrateur a grande frequence)

R. M. Berthier, *Revue des Materiaux* (Paris), No. 424, Jan. 1951, pp. 18-19

Reviewed by Phillip L. Melville

Standard vibrators utilizing an eccentric are handicapped by the problem of proper lubrication. A new patented device obviates it by utilizing the difference in specific gravities between aluminum and bronze in two half rings which create vibration. A 1/6 horsepower 3-phase motor turning at 10 to 12,000 rpm is used on laboratory specimens up to 20 kg.

Analysis of blast furnace slag in cements (Le dosage du laitier de haut fourneau dans les cements)

J. Brocard, *Revue des Materiaux* (Paris), No. 424, Jan. 1951, pp. 10-17

Reviewed by Phillip L. Melville

Testing laboratories often need to check the amount of slag in a cement. Qualitative analysis is the simplest, microscopic studies give fast results while quantitative analysis gives poor results. Separation by gravity, by electro-magnetism or lime content are unreliable. Microscopic studies are based on the vitreous and isotropic properties of slag and are correct within $\pm$ 1.5 percent.

Specification for concrete work

G. P. Manning, *Concrete and Constructional Engineering* (London), V. 46, No. 1, Jan. 1951, pp. 3-8

Reviewed by Glenn Murphy

The author compares the production of good concrete to the links on a chain, four of which are the materials. Additional links include handling materials, proportioning, mixing, transporting, placing, consolidating, curing and shrinkage. Brief comments are included concerning the importance of each in the making of good concrete. Among the pertinent observations is the following. "Curiously enough, in a country famous for good quality and sound craftsmanship, the English traditional outlook on concrete as a rough mixture to fill foundation trenches dies hard." The importance of specifications, research and standards is emphasized.

Observations from a year of travel

Walter B. Lenhart, *Rock Products*, V. 54, No. 1, Jan. 1951, pp. 131-142

Reviewed by Donald M. Agrimson

As a reporter, the author collected expressions and thoughts relating to trends within the rock industries. Some very interesting observations were made in his travels in which he visited rock crushing plants, sand processing plants, ready-mix plants, irrigation projects and concrete dams. He raises several questions on subjects that have each been answered by a variety of theories. For example, answers are sought to what affects durability of concrete, effects of heat on concrete, and what causes "growth" in concrete dams.

Laterally loaded plane structures and structures curved in space

Frank Baron and James P. Michalos, *Proceedings*, ASCE, Separate No. 51, V. 77, Jan. 1951, 33 pp.

Author's Summary

An analytical procedure is presented for determining the effects of loads normal to the plane of such structures as arches, bents and balcony girders. This procedure is called the "shear and torsion analogy." In addition, a procedure is presented for determining moments and shears in structures curved or segmental in space and continuous between two supports. These structures may be subjected to loads in any direction and to moments about any axis. The procedure is general, and consists of an extension, into space, of the shear and torsion analogy and of the column analogy.

Deterioration of concrete in the Herserud Lift (Betongskador pa Herserudshissen)

R. Hanson and S. Rosenstrom, *Betong* (Stockholm), V. 35, No. 4, 1950, pp. 301-310

Author's Summary

The Herserud Lift, Lidingö, near Stockholm, is a reinforced concrete structure whose useful life has been 35 years. The shortness of its life may be assumed to be due primarily to the method of construction. The consistency of the concrete was stiff, and the concrete was compacted by tamping in the forms. The compaction of the concrete was not sufficient to convert it into a homogeneous mass completely surrounding the reinforcement bars. The defective fixation of the reinforcement has contributed to the deterioration of the structure. In general, the

damage was caused by rusting of the reinforcement bars. Any primary defects due to freezing have not been observed. On the whole, the quality of the reinforcement steel and the composition of the concrete mix were satisfactory.

Some ideas on vacuum concrete (Quelques idees sur le vacuum concrete)

I. LEVIANT and E. DE LA SAYETTE, *Revue des Materiaux* (Paris), No. 423, Dec. 1950, pp. 373-7
Review by PHILLIP L. MELVILLE

A review of the principles and applications of vacuum concrete as used in the United States. Emphasis is on the reduction of the water-cement ratio to obtain a dry concrete with the following advantages: increased compressive and tensile strength, less shrinkage, less sensitivity to frost, better bond and watertightness. Excess mixing water is sucked out from the top through panels or from the side of the forms. It is claimed that perfectly watertight molds are not needed, that no cement is removed, no stratification occurs and no air voids are created. A shrinkage of 3 to 4 percent is observed.

Concrete construction in the Tyne tunnels

Concrete and Constructional Engineering (London), V. 46, No. 1, Jan. 1951, pp. 15-20
Reviewed by GLENN MURPHY

Two tunnels under the river Tyne are nearing completion after about four years' work. The pedestrian tunnel is 10 ft 6 in. in diameter and the cyclists tunnel is 12 ft in diameter. A vehicular tunnel has been proposed. Each of the tunnels is about 890 ft long and is reached at the end by escalators 170 ft long inclined at 30 degrees to the horizontal. The tunnels and escalator shafts are lined with cast-iron segments and filled between the inward projecting longitudinal flanges with concrete. The floors of the tunnels are constructed of precast reinforced concrete slabs supported on precast concrete beams and posts about 3 ft above the bottom of the tunnel. Floors are designed for a uniform load of 100 psf and the space beneath the floors carries ducts. The two entrance halls to the escalators involve elliptical concrete shell domes with major axes of about 60 ft and minor axes of about 55 ft. Construction details and photographs are included.

Sawdust concrete

Constructional Review (Sydney), V. 23, No. 8, Dec. 1950, pp. 25-27

Australian tests of concretes made with sawdust from indigenous woods showed that sawdust-cement mixtures are of rather uncertain and unpredictable strength, and the only safe procedure is to test the available sawdust to see whether the desired hardness can be produced. The addition of lime or calcium chloride is recommended to insure neutralization of extractable material which interferes with hardening of the cement. For certain species additional treatment is necessary.

Softwoods were found to be more generally suitable than hardwoods. For insulating concrete and for special purposes, mixes of about 1:3, 1:3½ or 1:4 gave best results. While richer mixes are stronger, they are heavier and are poor insulators. Leaner mixes have extremely low strength.

The amount of mixing water and method of adding it to the mix greatly affects strength. Previous soaking of the sawdust for 24 hours or longer increases the strength.

New findings, new precautions in regard to the decomposition of mortars and concrete in sea water (In French)

RENE DUBRISAY and HENRI LAFUMA, *Revue des Materiaux* (Paris), Special Issue, Oct., 1950, pp. 48-52
Reviewed by PHILLIP L. MELVILLE

Recent investigations indicate that the quality of concrete is due as much to good compaction, proper mix design and gradation as to chemical durability of the cement. Sea water has a mechanical action due to waves and erosion. Tides act through cycles of wetting and drying. Chemical action is by chlorides, sulfates, bicarbonates, alkalies. Sea water eliminates lime by diffusion. Hydrated lime is eliminated by hydrolysis which is only stopped by formation of carbonate, when this takes place slowly (solution of less than ½ g of *CaO* per l). Disintegration due to sulfates is a superficial action. Formation of sulfo-aluminates results in mortar disintegration. High-alumina cements do not seem subject to hydrolysis on account of recrystallization accompanied by expansion. Resistance against sea water may be obtained by reducing the alumina and lime to be replaced by a pozzolan. Slag cements are recommended and have proved

their value for harbor work. Best results are obtained if the slag cement is manufactured by the wet process. In addition the concrete should be dense, well placed and cured. Specifications are compared to those of American sulfate-resisting cements.

Manual on quality control of materials

American Society for Testing Materials, Philadelphia, 100 pp. $1.75

A new ASTM manual on quality control of materials takes the place of the widely used *Manual on Presentation of Data.*

Part 1 is a revision of the main section of the former manual and discusses the application of statistical methods, other problems of condensing the information contained in a single set of n observations, and presenting essential information in a concise form. Special attention is given to types of data such as would be gathered by individuals or committees with emphasis on the variability and the nature of frequency distributions of physical properties of materials.

Part 2 is a revision of Supplement A of the old ASTM manual and considers the problem of presenting limits to indicate the uncertainty of the average of a unique sample of n observations. Working rules are presented regarding the number of places to be retained in computation and presentation of average, standard deviations, and confidence limits. In this revision the generally accepted term "confidence limit" is introduced, and constants for computing 95 percent confidence limits are added.

Part 3 presents formulas, tables and examples useful in applying the control chart method of analysis and presentation of data mentioned above. This method requires that the data be obtained from several samples or that the data be capable of subdivision into subgroups on the basis of relevant engineering information. The principles of this part are discussed largely in terms of the quality of materials and manufactured products.

Bamboo reinforcement in portland cement concrete

H. E. Glenn, *Bulletin* No. 4, May 1950, Clemson Agricultural College of South Carolina, 172 pp.

The characteristics of bamboo as reinforcement for concrete were studied in rectangular and T-beams and in three experimental concrete structures. Seasoned and unseasoned bamboo specimens in the natural state or treated with asphalt emulsion or methylolurea were used whole or in split sections.

With an optimum reinforcement of 3 to 4 percent, failure loads of beams were increased 4 to 5 times but cracking occurred at loads similar to those for unreinforced members of the same dimensions. The condition of the bamboo had an important bearing on its behavior in the concrete. High percentages of dry bamboo, swelling through absorption of moisture, cracked the concrete. Seasoned bamboo brush-coated with asphalt emulsion developed greater load capacity than the untreated material although swell cracks developed with high percentages of longitudinal reinforcement.

Because bamboo-reinforced members deflect greatly before failure, design of structural members so reinforced is governed by the amount of deflection allowable. Due to the high deflection, bamboo-reinforced concrete members usually fail from other causes before the bamboo reinforcement reaches its ultimate tensile strength. A maximum of 3000 to 4000 psi allowable tensile stress in the bamboo must usually be imposed if deflection is to be kept under 1/360 of the span. Excessive deflection caused the failure of the beams in one of the test structures.

Resistance to fatigue of wires used in prestressed concrete (in French)

W. Soete and R. Vancrombrugge, *Annales des Travaux Publics de Belgique* (Brussels), V. 102, No. 5, Oct. 1949, pp. 513-533

Applied Mechanics Reviews Jan. 1951 (Massonnet)

The authors attempt to determine endurance limit of high-strength steel wires used in prestressed concrete, between stress limits used in practice. Tests were made on 5- and 7-mm diameter wires; minimum stress was chosen equal to ordinary prestressing stress, *i.e.*, 75 kg per sq mm; fatigue tests are limited to 2.10^6 cycles.

The machine used was a modified 30-ton Amsler pulsator. Pulsating stress was measured optically by Martens mirrors and electrically by SR-4 strain gages. Fixing of specimens in the machine gave much trouble as the wires always tended to break in the

grips because of the stress concentration they induced. The authors tried five different systems of grips, with circular or vee, toothed or smooth notches. The best system was that with vee toothed notches. Tests with 5-mm diameter wires all gave fractures in the grips, under a maximum stress varying from 95 to 105 kg per sq mm, according to the system of grips used. Tests on 7-mm wire were made on normal and heat-treated specimens, by using the best system of grips. Three specimens fractured in the grips and nine in the middle of the specimen under stresses from 100 to 105 kg per sq mm. According to this reviewer, these tests demonstrate that the weakest point in wires is the fastening itself; they raise the question of real resistance in these fastenings to fatigue loadings applied to prestressed beams, which should be cleared up experimentally.

Analysis and load tests of a long-span diagonally ribbed floor (Calcul et essai de charge d'un plancher a nervures en diagonal de grande portee)

M. P. Soutter, *Bulletin Technique De La Suisse Romande* (Lausanne), V. 76, No. 24, Dec. 2, 1950, pp. 325-330

Reviewed by J. Edmund Fitzgerald

The article describes the analysis and load testing of a diagonally ribbed concrete floor system used in the construction of a garage in Zurich, the "Bärenhof." The floor, which forms a quadrilateral approaching a square approximately 65 ft on a side, consists of ribs approximately 10 x 28 in. in section and 6 ft on centers both ways running at 45° to the sides. A 4-in. monolithic two-way slab rests on the grid beams. The floor is fixed along one side to an adjoining building and simply supported on a series of polished steel bearing plates on two sides. The third side is supported by an edge beam on four supports. The negative reactions at the corners are taken by wall brackets on the adjoining buildings. All four sides have a 10 x 28-in. edge beam. The cells next to the corners are filled with concrete to increase the plate action of the floor as a whole. An analytical solution employing finite differences was used. The initial deflection position was taken from the results of a 1/40 scale model test, in order to arrive at a solution rapidly. The analytical method presents nothing new over Marcus' original work on plates using finite differences H. Marcus, (*Elastische Gewebe*, Berlin, 1932). Full scale load tests indicated fair agreement between prototype, model and analysis. The analysis underestimated the restraining effects of the edge beams and was therefore on the conservative side.

After two years' service, the 7000 psi concrete floor shows no apparent cracking. The design load was about 250 lb per sq ft.

The author concludes that the behavior of the floor warrants the use of elastic theories for analysis and that an over-all economy of 10 to 15 percent was realized.

Producing gravel for McNary Dam

Walter B. Lenhart, *Rock Products*, V. 53, No. 11, Nov. 1950, pp. 66-70 and 94

Reviewed by Donald M. Agrimson

Aggregates for this huge project are obtained from basaltic river gravel deposits adjacent to the construction site.

The gravel screening plant itself is conventional, but features of the sand processing equipment are out of the ordinary in many respects. Four different sizes of sand were blended to make the one size concrete sand at the time of the author's inspection of the project. He expected that a fifth size would likely be added in the near future.

From the pit-run material is produced, in addition to the above mentioned sizes, four sizes of coarse aggregate—plus 3-in. to minus 6-in.; plus 1½-in. to minus 3-in.; plus ¾-in. to minus 1½-in.; and 4-mesh to minus ¾-in.

The small amount of plus 6-in. in the pit-run material is removed by a Symons vibrating grizzly. Following this stage some sand in the 30-mesh to minus 8-mesh size obtained from a near-by commercial producer is added since the pit-run is deficient in this size. This material is conveyed to four Symons wet vibrating screens where the four sizes of coarse aggregate are processed and minus ¼-in. material removed and delivered to the near-by sand plant.

The plus 16 mesh to minus ¼-in. is scalped over a wet Symons sand screen and conveyed directly to a storage bin over the blending belt. The minus 16 mesh flows from the Symons sand screen to three 8-spigot Dorrco sand sizers operating in parallel. These units operate continuously and automatically with a minimum of attention. With the above arrangement 8 sizes of sand could be produced.

Each of the three Dorrco products flows to Wemco sand machines for dewatering. From here each sand is sent to a hopper-bottomed tank located over the blending belt. By stockpiling in three piles a minimum drainage period of 72 hours is provided.

Prestressed concrete

GUSTAVE MAGNEL, Second edition, 1950, Concrete Publications Ltd., London, 300 pp. 16 shillings.

Reviewed by CHARLES C. ZOLLMAN

In this revised and enlarged second edition, the author presents, in his usual clear and simple manner, additional valuable design methods, tests and field applications developed since the publication of the first edition.

A new and easier design method for the important problem of end blocks is presented and illustrated with several numerical examples worked out in detail. The chapter on continuous structures has been rewritten and completed with numerical examples of continuous structures actually erected. Included are the computations on the 420-ft continuous two-span Selayn bridge.

A new chapter on the concept of prestressed rigid frames is included. Ordinary reinforced concrete columns are combined with prestressed girders to form a rigid frame. Principles are presented and illustrated with two examples of a single story rigid frame building. In the first example cast-in-place beams are used, in the second precast blocks.

The chapter on testing of beams has been expanded to include detailed laboratory test data of a continuous three-span girder tested to failure, and also detailed field test data on the Walnut Lane bridge girder.

Buckling of prestressed concrete rings is presented in Chapter 7, while results of tests on fatigue of wire are covered in Chapter 9. Worthwhile additions are the descriptions, plans, and photographs of numerous structures erected in the last two years in Great Britain, Belgium and the United States.

Those interested in precast work and who intend to enter the precast prestressed field will find a wealth of information on this particular subject in the last chapter.

The engineering profession will be grateful to the author for keeping it up to date on the progress made in this rapidly expanding field. Sufficiently detailed data are presented to enable the engineer to design prestressed structures of all types. The contractor will also find valuable information which will permit him to build such structures well.

Calcite in concrete (Calcit i betong)

O. H. STORM, O. M. FISKAA and E. STROMME, *Teknisk Ukeblad* (Oslo), V. 98, No. 2, Jan. 11, 1951, pp. 29-36

Reviewed by EIVIND HOGNESTAD

Papers pertaining to the use of calcite as an admixture in concrete were reviewed in the Nov. 1949 and Jan. 1950 ACI JOURNALS. The latter review pointed out that a considerable difference of opinion existed. A fact-finding committee consisting of the authors of the present report was appointed by the Norwegian government in March 1950 with the assignment to plan, supervise and report the tests necessary to obtain a clear understanding of the properties of calcite as an admixture.

The present report is based on a study of all existing pertinent information. The following conclusions are reached:

1. The existing test data indicate that the increased strength of mortars and concretes obtained by substituting pulverized calcite for part of a natural sand is due to physical phenomena, the so-called filler action. Calcite does not seem to possess superior properties in this respect as compared to other pulverized mineral aggregates used.

2. Nothing in the test data indicate a chemical reaction between the cement paste and calcite. The theory of formation of new calcite crystals is no more than a working hypothesis.

3. The resistance to some aggressives such as sulfates may, in certain cases, be increased by addition of calcite. This effect is, however, not only associated with calcite, but appears for many natural forms of calcium carbonate.

4. The test data indicate that the addition of finely pulverized calcite always reduces the resistance to freezing and thawing in the same manner as other pulverized mineral aggregates. It is possible that small additions of not too finely pulverized calcite may increase the frost resistance, but experimental proof is lacking.

5. It is not possible from existing evidence to judge whether durability in the tidal zone

may be improved significantly by the addition of coarsely or finely pulverized calcite.

It appears to this reviewer that the conclusions above should end the discussion of calcite. The authors recommend, however, that a further investigation be undertaken at Copenhagen since the Norwegian laboratories are involved in the controversy.

Development of a device for prestressing reinforced concrete structures (Dispositivo per la realizzazione di strutture di cemento armato precompresso)

RICCARDO MORANDI, *Giornale Del Cenio Civile* (Rome), V. 88, No. 3, Mar. 1950, pp. 131-136

The problem of anchoring wire strands in prestressed reinforced concrete structures (Il problema degli anchoraggi terminali nelle stutture in cemento armato)

GIUSEPPE RINALDI, *Giornale Del Genio Civile* (Rome), V. 88, No. 3, Mar. 1950, pp. 136-140
Reviewed by GENNARO MIANULLI

In the technique of prestressing reinforced concrete mechanically, the fundamental problem is the solution of the terminal anchorages. This problem, involving a few essential features, if solved in a different manner, leads to entirely different solutions, device and patent.

The essential features are the method and device used in tensioning the wires, the position of the cable, the maximum number of wire strands in a cable, the material constituting the anchorage, the cost of production and the sale value.

Each author states that the prestresssing device described in his article has been developed for use in Italy, is suitable to its economy and is built according to the essential features outlined above. Also, each author has compared the two devices, comments on the construction, merit and possible flaws.

The Morandi invention consists of a rectangular steel plate used mainly for pressure distribution. It has a large round opening on one side, where a steel cylinder is placed. The flanges of the cylinder are bolted to the plate. Slots are bored in both plate and cylinder, through which wires to be stretched by hydraulic jacks are passed. There is also a calibrating piece attached to the device which has the function of controlling pretensioning. A theoretical treatment and some computations explains the various parts of the anchorage and how it works as a unit.

Rinaldi's article discusses first the methods most used in Europe (Freyssinet and Magnel), then describes his method of prestressing structures. The device consists of a steel circular plate of appropriate diameter. Small slots are bored in the center of the plate and used to inject cement. Radially, a series of conical holes (whose axes are parallel with the axes of the circular plate) are perforated to receive the wire strands to be pretensioned by jacks.

Prefabricated apartment house construction (Bouwsystemen)

L. STOKLA, *Cement* (Amsterdam), No. 23-24, 1950, pp. 511-519
Reviewed by J. W. T. VAN ERP

Almost 30 years ago the high cost of traditional brick walls and wood floors induced the development of new systems, particularly in concrete construction for large housing projects. Although initial experiences with these new methods were not always favorable in low cost housing projects, improvements were gradually made. Ultimately satisfactory techniques evolved so that in the latest low cost housing projects prefabricated parts are used to a large extent, in combination with traditional and monolithic construction. The prefabricated concrete parts are mainly large sections of exterior walls, floor slabs and floor beams. Assembly is by tower crane, with no outside scaffolding, and use is being made of temporary steel bracing, to insure proper alignment and rigidity. The experience gained points to the following conclusions, bearing in mind that the cost of materials and mechanical equipment generally, in relation to labor costs involved, is much higher in European countries than it is here:

1. Over-all cost of prefabricated construction is still higher but in the last projects closely approaches that of traditional construction.

2. Costs are lowest if the modular unit adopted is small; this, however, is largely influenced by the high cost in upkeep and handling of large templates and steel molds. Small molds prove to have decided advantages.

3. Greater use of unskilled labor effects a saving in cost, particularly where no bricklayers and only few plasterers are needed to complete the job.

Progress in construction of concrete structures (Framsteg inom betongbyggnadstekniken)

H. Rusch, *Betong* (Stockholm), V. 35, No. 4, 1950, pp. 257-289

Author's Summary

The rivalry between structural steel and concrete has brought about great progress in the use of the latter in the last 10 years.

The cost of falsework and forms was attacked by applying scientific methods of design. Different systems of standardized falsework have been suggested; some examples are given. Also the design of improved "Melan" construction has led to lesser costs. Precast reinforced concrete elements have in many cases served well in place of wooden forms, principally in bridge construction. An instance is cited of a bridge being wholly prefabricated and afterwards moved into place; also a method of building large prestressed concrete bridges without falsework.

Principles are given for the design of prestressed concrete structures. It is pointed out that such design should be based not only on the allowable stresses but also show a given factor of safety against fracture. In most cases, then, the designer finds it impossible to utilize fully the strength of the steel when prestressing without a perfect bond between steel and concrete. For this reason prestressing with such bond is preferred today and used to produce smaller building parts, while big units, such as bridges in the first stage, are prestressed without bond between steel and concrete, and bond is introduced afterwards.

The qualities of prestressing steel and the different methods of anchorage are mentioned, also some German examples to show its possibilities.

Great progress has been made with precast building. Advantages and disadvantages of this method are described. The method may be used for light roofs on tall buildings, in which case the designer should favor a solution with a great number of identical units to introduce, if possible, mass production. Examples, such as framework, roofs and bridges are given to stress the many possibilities, but it should be understood that lesser savings are to be had with bridges than with other structures.

A special method of precasting has been used and is described in detail. This method uses concrete matrix molds that are made easily at the building site and made to sustain heating. One finished building unit may be produced every day with each matrix mold.

Scottish hydro-electric works

Concrete and Constructional Engineering (London), V. 45, No. 12, Dec. 1950, pp. 419-425

Reviewed by Glenn Murphy

Details of concrete work in some of the structures of two hydro-electric works in Scotland are described. Photographs are given of Loch Sloy Dam and Inveruglas generating station. Plans for the dam, a buttress concrete structure 1160 ft long, were described in Mar. 1947 and Dec. 1948 *Concrete and Constructional Engineering*. The dam contains 200,000 cu yd of concrete and has a maximum height of 165 ft. The generating station which is about two miles from the dam was put in operation in Oct. 1950. Precast concrete slabs covered the 9½ in. reinforced concrete external walls. Slabs are anchored to the wall but separated by a 1-in. space later filled with grout. Natural colored aggregate was used for the slabs which were subsequently scrubbed to expose the aggregate.

Clunie Dam previously described in Jan. and Feb. 1949 *Concrete and Constructional Engineering* is now completed. The gravity structure, 380 ft long with a maximum height of 70 ft and a maximum width of 90 ft contains about 37,000 cu yd of concrete. The 2-mile tunnel horseshoe to the generating station has an equivalent diameter of 23 ft and is the largest concrete lined water tunnel in Britain. Details of the concrete work on the generating station are described.

Precast slabs 22 in. deep are used in construction of the Pitlochry generating station. The walls are reinforced concrete. The dam is about 480 ft long and 50 ft high and contains about 43,000 cu yd of concrete. Construction included a 300-ft reinforced concrete bridge across the Tummel River. The center span of 94 ft consists of two cantilevers and a 50-ft suspended span. End spans are 77 ft long; the piers are supported on piles. The fish ladder at Pitlochry is about 1000 ft long, consisting of 33 compartments about 25 ft by 14 ft with a rise of 18 in. between compartments. The compartments are connected by a submerged pipe or orifice. Somewhat similar fish ladders are used at Clunie.